Influence Line Computations for Structures with Members of Varying Flexural Rigidity Using the Electronic Digital Computer

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Introduction

The analysis of a structure carrying prescribed loading and subjected to given support conditions consists in finding a set of forces or displacements satisfying the basic requirements of equilibrium and continuity. In simple structures it is generally possible to satisfy these conditions simultaneously whereas in more difficult cases the principle of superposition may be used after satisfying the conditions separately.

Two distinct methods of approach are possible depending on the order in which the conditions are fulfilled. In the 'flexibility,' or 'force,' method the structure is made statically determinate by the introduction of releases at suitably chosen points, thus equilibrium is maintained whilst continuity is lost. The structural analysis is designed to restore the lost continuity and in this case the unknowns are forces (or bending moments etc.), a familiar example of the method being the three-moment theorem.

In the 'stiffness,' or 'displacement,' method the unloaded structure is considered clamped at the joints so that all the joint displacements, linear and angular, are zero. On applying the loads to the structure external forces (or moments) are required at the joints to maintain zero joint displacements. The analysis is completed by calculating the joint displacements necessary to remove the unwanted external actions. Thus the unknowns in this case are the joint displacements, a familiar example of the method being slope-deflexion.

For structures in which forces and displacements are linearly related the analysis will result in a set of linear simultaneous equations with either forces or displacements constituting the unknowns. In the 'force' method the number of equations will equal the 'statical indeterminacy' number[1] and in the 'displacement' method the 'kinematical indeterminacy number'[2].

Both approaches to structural analysis can be expressed conveniently in matrix form and are thus eminently suitable for solution on a digital computer. The choice of approach is influenced by several factors. For hand computation the analyst would almost certainly select the method which involved the smaller number of unknowns and so keep the number of equations to a minimum. For machine computation the important factors are the storage capacity required and ease of setting up the data for the machine.

A direct advantage of the matrix method of formulating the analysis is in the existence of standard routines for performing the matrix operations. In the case of the Ferranti Mercury computer single instructions to the machine will perform all the matrix operations required in the analysis. For example the statement

$$A' = \varphi\, 26(B',C',20,20,30)$$

instructs the machine to perform the operation of matrix multiplication, $A_{(20 \times 30)} = B_{(20 \times 30)}$. $C_{(30 \times 20)}$, where the dimensions of the matrices are indicated by the suffices. The primed letters A', B' and C' denote the storage locations in the computer of the first elements of the respective matrices.

Other instructions are available for all the usual matrix operations of addition, subtraction, transposition, inversion and so on. The writing of a programme is thus greatly simplified if the calculation can be broken down into a series of matrix operations. Furthermore the computer can be used to assemble the matrices and thus reduce the effort required in preparing data for the problem.

Structures in which the flexural rigidities of the members vary within their lengths, although not unusually difficult to analyse, provide the designer with a large amount of computation and it has been the author's intention to produce a programme specially suited to structures of this kind. The need to calculate influence line ordinates for structures with varying flexural rigidity further increases the amount of computation and the programme which is to be described is designed primarily to deal with this problem. The flexibility influence coefficient method is used with a matrix treatment similar to that used by Henderson, Edwards and Munro[2] for the Pegasus computer. The present programme is written for the Mercury computer.

Theory

Consider a structure statically indeterminate to the nth degree and introduce n releases to make a statically determinate system (the 'reduced structure'). We may express the displacements in terms of the redundant actions, x, and use the principle of superposition to obtain,

$$\left.\begin{aligned}
\delta_1 &= u_1 + x_1 f_{11} + x_2 f_{12} + \ldots + x_n f_{1n} \\
\delta_2 &= u_2 + x_1 f_{21} + x_2 f_{22} + \ldots + x_n f_{2n} \\
&\cdots \\
\delta_n &= u_n + x_1 f_{n1} + x_2 f_{n2} + \ldots + x_n f_{nn}
\end{aligned}\right\} \quad \ldots \quad (1)$$

where,

δ_n = displacement discontinuity in the actual structure at the position of the nth release.

u_n = displacement discontinuity in the reduced structure at the nth release due to the applied loading.

f_{mn} = displacement discontinuity at the mth release due to unit action at the nth release in the reduced structure.
= flexibility influence coefficient.

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If bending distortions alone are considered then,

$$f_{mn} = \int \frac{m_m m_n ds}{EI}$$
$$\text{and, } u_n = \int \frac{m_n m_o ds}{EI} \quad \cdots (2)$$

in which,

m_n = bending moment distribution in the reduced structure due to unit action replacing the nth redundancy.

m_o = bending moment distribution in the reduced structure due to the applied load system.

The integrations represented by equations (2) are to be evaluated over the whole structure. We consider the case where EI varies within the lengths of the members and use Simpson's rule to make the summations.

The whole solution can be performed using matrix methods with the orders of the matrices defined by combinations of the three parameters,

n = number of unknowns (statical indeterminacy number)

v = number of integration points in the structure, and,

w = number of loading cases, i.e. number of positions of a unit load if influence lines are required.

If there are no discontinuities in the actual structure at the positions of the releases then $\delta = O$ throughout and the basic equation for the structure is,

$$FX = -U \quad \cdots (3)$$

where,

F = flexibility matrix of the reduced structure. F, a symmetric matrix of order n, is independent of the applied load system but dependent on the release system adopted.

X = a rectangular $(n \times w)$ matrix of unknown forces (or moments).

U = a rectangular $(n \times w)$ matrix of load functions.

$$F_{(n \times n)} = \begin{bmatrix} f_{11} & f_{12} & \cdots & f_{1n} \\ \cdot & & & \cdot \\ \cdot & & & \cdot \\ f_{n1} & f_{n2} & \cdots & f_{nn} \end{bmatrix} \quad \cdots (4)$$

$$X_{(n \times w)} = \begin{bmatrix} (x_1)_1 & (x_1)_2 & \cdots & (x_1)_w \\ \cdot & & & \cdot \\ \cdot & & & \cdot \\ (x_n)_1 & (x_n)_2 & \cdots & (x_n)_w \end{bmatrix} \quad \cdots (5)$$

$$U_{(n \times w)} = \begin{bmatrix} (u_1)_1 & (u_1)_2 & \cdots & (u_1)_w \\ \cdot & & & \cdot \\ \cdot & & & \cdot \\ (u_n)_1 & (u_n)_2 & \cdots & (u_n)_w \end{bmatrix} \quad \cdots (6)$$

The f and u functions in (4) and (6) are as defined in equations (2).

In order to compute the elements of the F and U matrices a matrix M of order $(v \times n)$ is introduced the elements of which are the values of $m_1, m_2, \ldots m_n$ at the v integration points, thus,

$$M_{(v \times n)} = \begin{bmatrix} (m_1)_1 & (m_2)_1 & \cdots & (m_n)_1 \\ (m_1)_2 & (m_2)_2 & \cdots & (m_n)_2 \\ \cdot & & & \cdot \\ \cdot & & & \cdot \\ (m_1)_v & (m_2)_v & \cdots & (m_n)_v \end{bmatrix} \quad \cdots (7)$$

The summations required by equation (2) can be performed by matrix multiplications using a diagonal matrix D. The diagonal elements of D are the values of $\frac{h}{3EI}$ (Simpson's rule coefficient) at each of the v integration points where h is the step length appropriate to the range of integration. The off-diagonal elements of D are zero.

It should be noted that an even number of equal intervals is required in each range of integration (usually each member) in conformity with the Simpson's rule requirements.

Matrix M is pre-multiplied by matrix D giving,

$$DM = \begin{bmatrix} D_1(m_1)_1 & D_1(m_2)_1 & \cdots & D_1(m_n)_1 \\ D_2(m_1)_2 & D_2(m_2)_2 & \cdots & D_2(m_n)_2 \\ \cdot & & & \cdot \\ \cdot & & & \cdot \\ D_v(m_1)_v & D_v(m_2)_v & \cdots & D_v(m_n)_v \end{bmatrix} \quad \cdots (8)$$

Matrix M is then pre-multiplied by the transpose of DM thus,

$$(DM)^T M = \begin{bmatrix} \sum_{p=1}^{v}(Dm_1m_1)_p & \sum_{p=1}^{v}(Dm_1m_2)_p & \cdots & \sum_{p=1}^{v}(Dm_1m_n)_p \\ \sum_{p=1}^{v}(Dm_2m_1)_p & \sum_{p=1}^{v}(Dm_2m_2)_p & \cdots & \sum_{p=1}^{v}(Dm_2m_n)_p \\ \cdot & & & \cdot \\ \cdot & & & \cdot \\ \sum_{p=1}^{v}(Dm_nm_1)_p & \sum_{p=1}^{v}(Dm_nm_2)_p & \cdots & \sum_{p=1}^{v}(Dm_nm_n)_p \end{bmatrix} \quad \cdots (9)$$

Thus, from equations (2), (4) and (9),

$$F = (DM)^T M \quad \cdots (10)$$

The elements of the U matrix, equations (3) and (6), are computed in a similar way by introducing a matrix M_o of order $(v \times w)$. The elements of M_o are the values of bending moment in the reduced structure at the v integration points due to the w applied load systems, thus,

$$M_{o\,(v \times w)} = \begin{bmatrix} (m_o)_{1,1} & (m_o)_{1,2} & \cdots & (m_o)_{1,w} \\ (m_o)_{2,1} & (m_o)_{2,2} & \cdots & (m_o)_{2,w} \\ \cdot & & & \cdot \\ \cdot & & & \cdot \\ (m_o)_{v,1} & (m_o)_{v,2} & \cdots & (m_o)_{v,w} \end{bmatrix} \quad \cdots (11)$$

It follows that,

$$(DM)^T M_o = \begin{bmatrix} \sum_{p=1}^{v}[Dm_1(m_o)_1]_p & \sum_{p=1}^{v}[Dm_1(m_o)_2]_p & \cdots & \sum_{p=1}^{v}[Dm_1(m_o)_w]_p \\ \sum_{p=1}^{v}[Dm_2(m_o)_1]_p & \sum_{p=1}^{v}[Dm_2(m_o)_2]_p & \cdots & \sum_{p=1}^{v}[Dm_2(m_o)_w]_p \\ \cdot & & & \cdot \\ \cdot & & & \cdot \\ \sum_{p=1}^{v}[Dm_n(m_o)_1]_p & \sum_{p=1}^{v}[Dm_n(m_o)_2]_p & \cdots & \sum_{p=1}^{v}[Dm_n(m_o)_w]_p \end{bmatrix} \quad \cdots (12)$$

Thus, from equations (2), (6) and (12),

$$U = (DM)^T M_o \quad \cdots (13)$$

The computer assembles the matrices D, M and M_o and performs the matrix multiplications of equations (10) and (13), the solutions are then, from equation (3),

$$X = -F^{-1}U \quad \cdots (14)$$

Outline of programme

The programme is written in a general way in terms of parameters which are the first numbers to appear on the data tape. Thus the computer is informed of the orders of the matrices involved in a particular problem and allocates the storage accordingly. The order of carrying out the various operations is as follows,

(1) Input the three parameters n, v and w.

(2) Allocate storage.

(3) Input values of h for each range of integration and EI values at each point of integration.

(4) The computer now calculates the diagonal elements of D generating the Simpson's rule coefficients automatically.

(5) Input characteristic numbers and principal ordinates of the m and m_0 diagrams (see later under heading 'arrangement of data').

(6) The computer now calculates intermediate values of bending moment and assembles the M and M_0 matrices.

(7) Form DM and $(DM)^T$

(8) Form $F = (DM)^T M$ and print F.
Form $U = (DM)^T M_0$

(9) Invert F and print inverse matrix.

(10) Form $X = -F^{-1}U$ and print X.

The storage capacity of the computer imposes a limit on the size of structure that can be dealt with in this way. The actual limits depend on combinations of the three parameters n, v and w and may be expressed, for the Mercury computer, as follows,

$$v^2 + 3nv + vw + 2n^2 + 2nw \leq 10{,}752$$

For example, if $n = 7$ redundancies and $v = 79$ integration points then $w \leq 29$ loading cases.

A further limit is imposed by the programme in its present state. The total number of separate ranges of integration should not exceed twenty. In fact it would not be difficult to raise this limit but it is not thought necessary since the value of v for such a structure would probably exceed the storage capacity.

The printing of the F and F^{-1} matrices can be suppressed if desired. These matrices were required in developing the programme for checking purposes but would not normally be required by a designer.

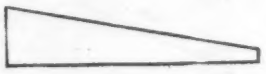
Arrangement of the data

This is conveniently illustrated by reference to the data for example 1 shown in Table 1. The process of preparing and assembling the data can be carried out in the following steps. Select a release system and number the redundancies 1 to n. Divide each range of integration (generally each member of the structure) into an even number of steps and number the points from 1 to v over the whole structure. In counting v, end points common to two ranges of integration are included in both. Sketch the m_0 diagrams. Each of the diagrams represents the bending moment distribution in the reduced (statically determinate) structure due to a unit load in a particular position, thus w diagrams are required. Sketch the m diagrams. These represent bending moment distributions in the unloaded reduced structure due to unit actions replacing each redundancy in turn. The principal values only are required, the computer will calculate the intermediate values. Select a suitable sign convention for the diagrams; drawing the diagrams on the tension sides of members is a convenient method.

The data tape can now be prepared. The first three numbers on the tape are the values of the parameters n, v and w for the problem. These are followed by the number of ranges of integration, 4 in example 1, and the value of h appropriate to each range. These values are followed by the numbers of integration points within each range, in this example 7, 9, 9 and 7, and it should be noted that these numbers must be odd since Simpson's rule is used. Then follow the EI values, actual or relative, at the integration points listed in the correct order with common end values included in each range meeting at the point. Lastly information about the m and m_0 diagrams is assembled as follows. For the first redundancy and for each range of integration in turn give the characteristic bending moment diagram numbers 1, 2 or 3.

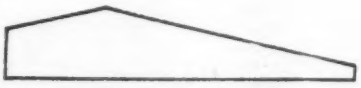
1 Indicates zero bending moment throughout the range.

2 Indicates a trapezoidal bending moment diagram



and

3 Indicates a diagram of the form



This is followed by, for type 2, the two end values, for type 3 the value at the beginning of the range, the number of the point where the peak lies (counting from the beginning of the range,) the value at the peak and the value at the end point in the range. This is repeated for the second and subsequent redundancies, i.e. n times in all. The m_0 diagrams are dealt with in the same way for each unit load position in turn, i.e. w times. If a diagram is of type 1 (zero bending moment throughout the range) then no additional data is required.

It is of course vital to get the data in the correct order on the tape. The arrangement used in the print-out of the data for example 1 is convenient for checking purposes but is not necessary, it is however essential that the numbers are given in the correct order, each number being separated from adjacent numbers by at least two spaces or placed on a new line.

Example 1

This example is a four-span continuous beam with parabolic soffits, subjected to vertical restraint only at the supports. The redundant elements are taken to be the bending moments at the interior supports. Figure 1 shows a diagram of the structure together with the appropriate m and m_0 bending moment diagrams. The structure is symmetrical and fourteen positions are taken for the unit load on one half of the structure. The data and results for the problem are shown in Table 1. The results appear in 'floating decimal' style, i.e. the exponent after the comma is the power of 10 by which the number must be multiplied. The F and F^{-1} matrices are printed out row by row, i.e. the first three numbers are the first row of the F matrix.

Data		Results			
3 32 14 4 15 20 20 15 7 9 9 7	parameters	2·7033, 1 1·5645, 1 1·1832, –75			
1 1·125 1·60 2·59 4·60 8·49 15·63 15·63 6·30 2·61 1·32 1 1·32 2·61 6·30 15·63 15·63 6·30 2·61 1·32 1 1·32 2·61 6·30 15·63 15·63 8·49 4·60 2·59 1·60 1·125 1	relative *EI* values	1·5645, 1 4·1984, 1 1·5645, 1 1·1832, –75 1·5645, 1 2·7033, 1 5·1020, –2 –2·4239, –2 1·4028, –2 –2·4239, –2 4·1884, –2 –2·4239, –2 1·4028, –2 –2·4239, –2 5·1020, –2	*F* F^{-1}		
2 2 1 1 0 1 1 0 1 2 2 1 0 1 1 0 1 1 2 2 0 1 1 0	m	5·0501, 0 7·8326, 0 9·0208, 0 7·2541, 0 4·3782, 0 1·3373, –77 1·3401, 1 2·3279, 1 2·9866, 1 2·7242, 1 2·1048, 1 1·3244, 1 5·9941, 0 1·3373, –77	x_1	–2·3993, 0 –3·7212, 0 –4·2857, 0 –3·4463, 0 –2·0801, 0 –1·3989, –77 3·1353, 0 7·6228, 0 1·2898, 1 1·7948, 1 2·0646, 1 1·6439, 1 9·6427, 0 –1·3989, –77	x_2
3 1 1 1 0 2 –12·5 0 3 1 1 1 0 3 –20 0 3 1 1 1 0 4 –22·5 0 3 1 1 1 0 5 –20 0 3 1 1 1 0 6 –12·5 0 1 1 1 1 1 3 1 1 0 2 –17·5 0 1 3 1 1 0 3 –30 0 1 3 1 1 0 4 –37·5 0 1 3 1 1 0 5 –40 0 1 3 1 1 0 6 –37·5 0 1 3 1 1 0 7 –30 0 1 3 1 1 0 8 –17·5 0 1 1 1 1	m_0	1·3885, 0 2·1536, 0 2·4803, 0 1·9945, 0 1·2038, 0 1·5488, –77 –1·8145, 0 –4·4116, 0 –7·4645, 0 –1·0387, 1 –1·1948, 1 –9·5140, 0 –5·5805, 0 1·5488, –77	x_3		

Table 1

Data and results for example 1.

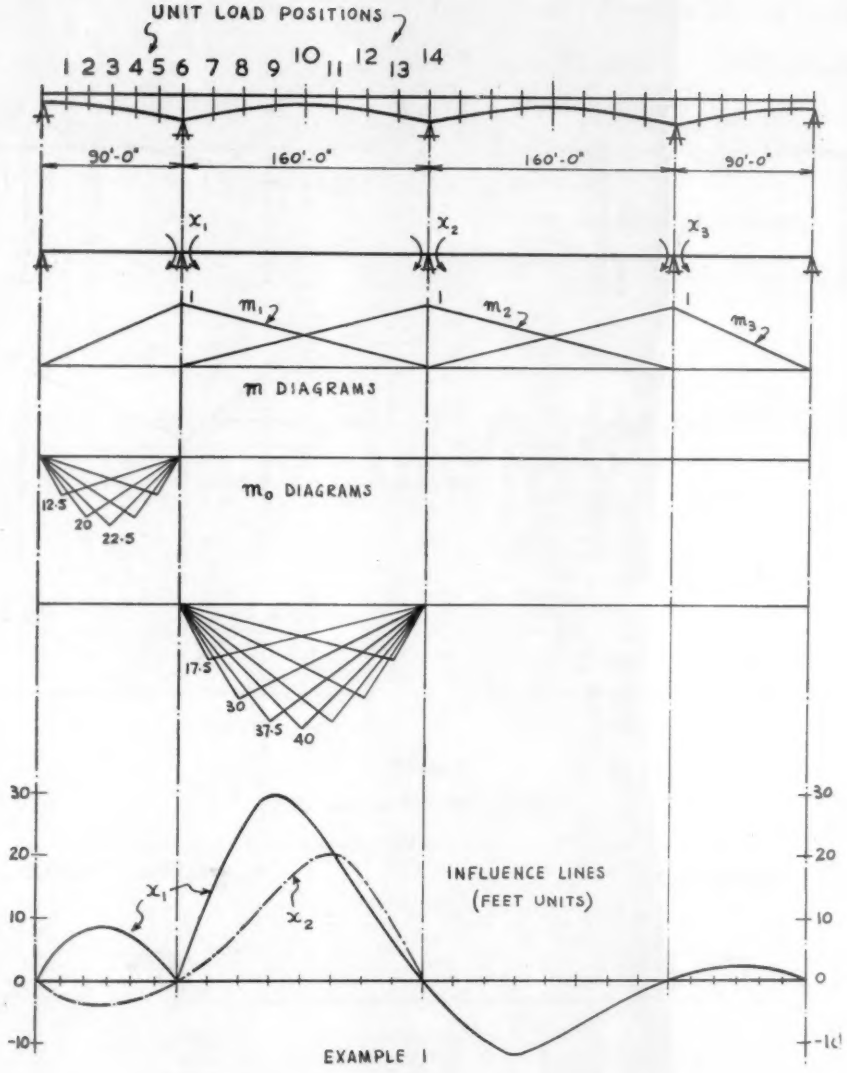
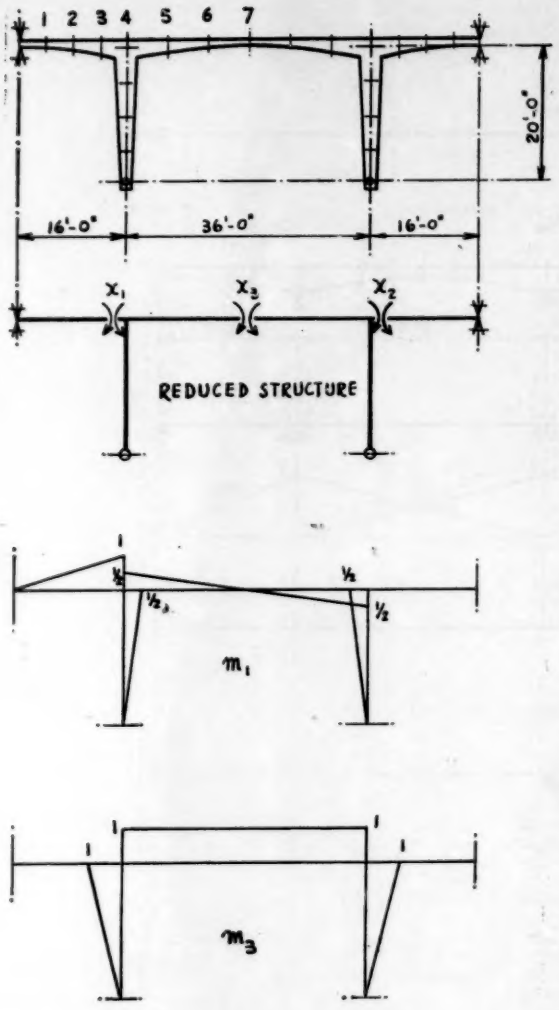
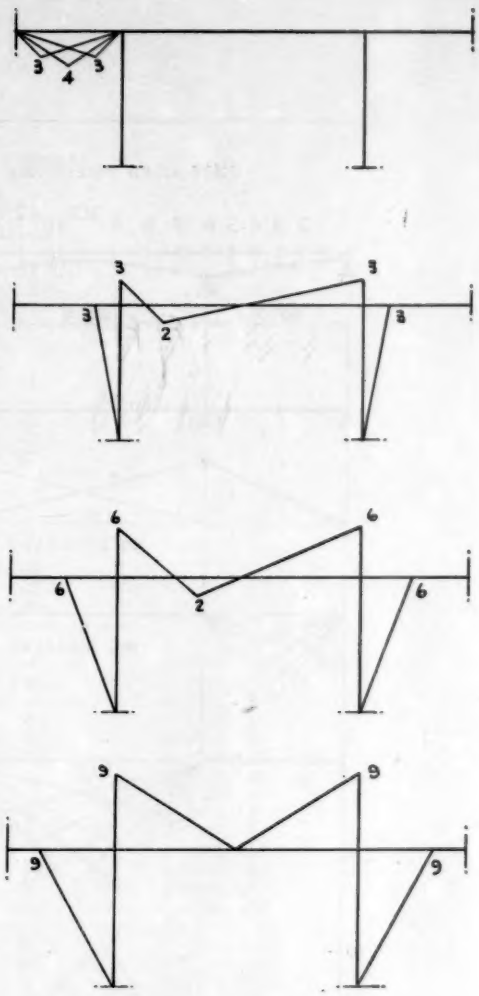

UNIT LOAD POSITIONS
1 2 3 4 5 6 7 8 9 10 11 12 13 14
90'-0"
160'-0"
160'-0"
90'-0"
X1
X2
X3
1
m1
1
m2
1
m3
m DIAGRAMS
m0 DIAGRAMS
12·5
20
22·5
17·5
30
37·5
40
30
20
10
0
-10
X1
X2
INFLUENCE LINES
(FEET UNITS)
EXAMPLE 1

Fig. 1.



EXAMPLE. 2

Fig. 2.



m_o DIAGRAMS, EXAMPLE 2

Fig. 3.

Example 2.

This example is a rigid-frame bridge structure, the beams having parabolic soffits and the columns straight line tapers. The columns are hinged at the feet and the beams subjected to vertical restraint only at their outer ends. The structure has three redundancies taken to be the internal bending moments in the outer spans at their connexions with the columns and at the mid-span of the structure. Figure 2 shows an outline of the structure together with the relevant m and m_o bending moment diagrams. The data and results for this example are shown in Table 2.

Use of Simpson's Rule and accuracy of the results

A discontinuity in the EI variation or in one of the bending moment diagrams will introduce an error if it occurs at one of the even numbered points. This situation occurs at alternate unit load positions in the examples. The difficulty can be avoided if the intervals are halved throughout and the unit load positions retained. The unit load then automatically falls at odd numbered points. To investigate the errors involved example 1 was repeated with halved intervals and the results are shown in Table 3. The errors at the even numbered points are evident but are not particularly serious in this case.

Data		Results	
3 27 7	parameters	2·5507, 0	
5 4 6 4 5 5		−2·8958, −1	
5 7 5 5 5		−8·7108, −1	
1·28		−2·8958, −1	
1·28		2·5507, 0	F
1·78		−8·7108, −1	
3·98			
1000		−8·7108, −1	
1000		−8·7108, −1	
3·12		2·0430, 1	
1·44			
1		4·0468, −1	
1·44		5·2601, −2	
3·12		1·9497, −2	
1000			
1000		5·2601, −2	
3·98		4·0468, −1	F^{-1}
1·78	relative EI	1·9497, −2	
1·28	values		
1·28		1·9497, −2	
1·28		1·9497, −2	
2·61		5·0610, −2	
5·36			
8·55		2·2776, 0	
14·20		2·8690, 0	
1·28		2·2479, 0	
2·61		−5·7860, −77	x_1
5·36		2·6225, 0	
8·55		3·1396, 0	
14·20		2·3537, 0	
2 2 1 2 2 0 1 0·5 −0·5 0 −0·5 0 −0·5		2·9605, −1	
m 1 2 2 2 2 −0·5 0·5 1 0 0 −0·5 0 −0·5		3·7292, −1	
1 2 1 2 2 1 1 0 1 0 1		2·9219, −1	
		−5·7860, −77	x_2
3 1 1 1 1 0 2 −3 0		−4·3683, −1	
3 1 1 1 1 0 3 −4 0		6·3190, −1	
3 1 1 1 1 0 4 −3 0		2·3537, 0	
m_0 1 1 1 1 1			
1 3 1 2 2 3 2 −2 3 0 3 0 3		1·0973, −1	
1 3 1 2 2 6 3 −2 6 0 6 0 6		1·3823, −1	
1 3 1 2 2 9 4 0 9 0 9 0 9		1·0830, −1	
		−1·4993, −77	x_3
		−1·6322, −1	
		−1·1051, 0	
		−2·8904, 0	

Table 2

Data and results for example 2.

x_1	x_2	x_3
4·7382, 0	−2·2556, 0	1·3076, 0
7·8620, 0	−3·7427, 0	2·1696, 0
8·6526, 0	−4·1191, 0	2·3878, 0
7·2558, 0	−3·4542, 0	2·0023, 0
4·1965, 0	−1·9978, 0	1·1581, 0
−1·4239, −77	6·9293, −78	−2·1269, −77
1·2997, 1	3·3082, 0	−1·9178, 0
2·3260, 1	7·6226, 0	−4·4188, 0
2·8595, 1	1·2998, 1	−7·5350, 0
2·7341, 1	1·8012, 1	−1·0441, 1
2·1000, 1	1·9673, 1	−1·1404, 1
1·3244, 1	1·6425, 1	−9·5217, 0
6·1634, 0	9·3144, 0	−5·3995, 0
−1·4239, −77	6·9293, −78	−2·1269, −77

Table 3

Results for example 1 with step lengths halved throughout.

Condition of the equations and choice of release system

The condition of the equations can be assessed by examining the flexibility matrix F. For well-conditioned equations the elements on the principal diagonal should be large in comparison with the off-diagonal elements. In order to achieve this the release system should be chosen in such a way that there is a limited overlap of the m diagrams.[3] The F matrix may be examined for condition by normalising by division of the elements of the ith row ($i = 1,2,..n$) by $\sqrt{\sum_{j=1}^{n} f^2_{ij}}$ and then evaluating the determinant of F. This is then compared with ± 1 and relatively small values of $|F|$ indicate ill-condition. A subsidiary programme performs this calculation and it is found that values of $|F|$ for the examples 1 and 2 are respectively 0·377 and 0·844. A more refined error analysis may be made using the method described by McCalley[4].

The programme has been modified to allow for an imposed displacement discontinuity in the structure at one of the releases. The programme will deal only with a constant numerical value of yield (linear or angular) and not with a value which is a function of the redundancy.

The programme described has been developed with a particular type of problem in mind and some sacrifices of generality have been made to keep the data preparation down to a minimum. The effects of axial forces are neglected and the programme will not deal with curved bending moment diagrams. These effects could be included without undue difficulty but would reduce the available storage capacity and perhaps the usefulness of the programme.

Both examples quoted are of symmetrical structures for which complete influence lines are obtained for unit load positions covering one symmetrical half. In unsymmetrical cases the unit load positions must of course cover the whole structure.

Acknowledgement

The work described was carried out on the Mercury computer at the University of London Computer Unit and the author is grateful to the Director Dr. R. A. Buckingham and his staff for their help and advice.

References

1. Henderson, J. C. de C., and Bickley, W. G., "Statical Indeterminacy of a Structure," *Aircraft Engineering*, December 1955.
2. Henderson, J. C. de C., Edwards, A. D., and Munro, J., "Influence Coefficient Methods of Linear Structural Analysis," *Civil Engineering and Public Works Review*, April and May 1960.
3. Argyris, J. H., "Energy Theorems and Structural Analysis," *Aircraft Engineering*, February 1955.
4. McCalley, R. B., "Ill-conditioned Equations and Matrices," *Conference on Electronic Computation, American Society of Civil Engineers*, 1958.

Discussion

The Council would be glad to consider the publication of correspondence in connexion with the above paper. Communications on this subject intended for publication should be forwarded to reach the Institution by December 31st, 1961.

Book Reviews

Calcul a la Rupture et Plasticité des Constructions by A. R. Rianitsyn. Translated into French from Russian by N. V. Sobarnitsky. Foreword by R. L'Hermite. (Paris : Eyrolles, 1959). 284 pp. Fr. 4400.

This excellently produced book gives a clear scientific presentation of many aspects of Ultimate Load theory, and the implications of inelastic deformations as they may occur in various structural units and systems. The influence of plasticity on the strength of beam sections in bending, of initial stresses and repeated loading is considered. Possible modes of failure of elasto-plastic continuous beams are discussed, and the ultimate resistance of eccentrically loaded ties and struts, with the special case of the arch. General equations for elasto-plastic systems are presented and their application to simple cases. Immediate use might be made of the theory in the case of structural steel where the characteristics of deformation are well defined, but in the case of a brittle and more complex material such as reinforced concrete, many test results and semi-empirical formulae will be required in conjunction with the basic theory, to express rotation values at hinges and their possible limits.

The influence of Poisson's ratio on the ultimate strength of a tension member is investigated. The calculation of the ultimate strength of slabs by crack line theory as developed by Johansen and Gvozdev is explained. Other important subjects associated with Ultimate Theory such as inelastic behaviour at low stress, creep and the probability of error in calculations are given basic mathematical treatment. This is a book for research and academic structural engineers. It is realistic in its recognition of the true behaviour of materials and expresses mathematically and diagramatically those characteristics, which depart radically from the normally assumed elastic properties. For the practising engineer it presents a true fundamental concept of structural behaviour from no-load to failure, which is the beginning of wisdom. Research must now follow in the case of the more complex materials to establish simple rules for calculating within safe limits, the inelastic rotations adjacent to critical sections both at working and ultimate load.

The Use of Welding in Steel Building Structures, by G. Bernard Godfrey. (London : B.C.S.A. Publication No. 14/1960). 11 in. × 8½ in. 35 pp. 3s. 6d.

This publication is a revised edition of B.C.S.A. Brochure No. 6, originally published in 1952. The text and illustrations have been revised in conformity with the requirements of B.S.449/1959, "The Use of Structural Steel in Building." The subjects dealt with are fillet and butt welds, welded details and built-up columns and complete designs are given for a welded plate girder and a crane girder.

The Effective Width of a Composite Tee-Beam Flange

by

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and

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Synopsis

Composite construction, using rolled steel beams and concrete floor slabs, has been used for construction of highway bridge decks and building floors for a considerable number of years. So far as the Authors are aware, no comprehensive experimental investigation has been undertaken to determine the effective width of floor slab which may be assumed to act as the compression flange in such beams. The following paper describes laboratory tests carried out on four small scale composite beam and slab floors, from which the effective flange width of the composite tee-beams has been determined in each case.

Details of the experimental work are briefly given in the paper. The experimentally determined effective flange widths are compared with values obtained from existing theory and also with those recommended by some design authorities. Differences occur between the flange widths determined experimentally and those obtained by analytical methods, but both indicate that the effective width of a composite tee-beam flange decreases with increase in the ratio of beam spacing over beam span ($2b/L$). The investigation also shows that analytical values of effective widths are smaller than those obtained experimentally.

Introduction

It is generally accepted that when a simply supported steel beam has its top flange adequately bonded to a concrete slab, greater load carrying capacity results through composite action of the steel and concrete. Experimental investigations have been carried out in various countries to determine the relative merits of different types of shear connectors used for bonding the beam flange to the slab, and to evolve design methods suitable for such construction[1,2,3,4]. As a result of these studies, many highway bridge and building structures have been designed and constructed in accordance with the principles of composite action.

Since a composite beam gains its advantage from utilization of the concrete slab as a compression flange, economical design results only when full use is made of the slab. When the beam spacings are large, shear lag effect may become important, and the width of tee-beam flange assumed for design purposes needs further consideration. Although recommendations[5] and specifications[6] for the effective width of composite tee-beam flanges already exist, the Authors have felt that such proposals should not be accepted fully unless endorsed by further experimental investigations.

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With this object in view a laboratory investigation was undertaken on four small scale slab and I-beam floors to study the effective width of slab which could safely be assumed to act integrally with the steel I-beams.

Experimental Investigation

(a) *Test Specimens*

Since the primary object of the investigation was to study the effective width of composite tee-beam flanges, the test specimens were designed as slabs resting on top of a number of evenly spaced, simply supported rolled steel joists of 4 in. × 3 in. × 10 lb. section. The slabs were effectively anchored to the steel joists by means of continuous double spiral shear-connectors welded to the top flanges of the joists and embedded in the slab concrete. In this way the test specimens could be assumed to consist of a series of composite tee-beams, each made up of a rolled steel joist and a certain width of slab forming its top flange.

Details of the test specimens are given in Table 1, and Fig. 1 shows one of the specimens during construction.

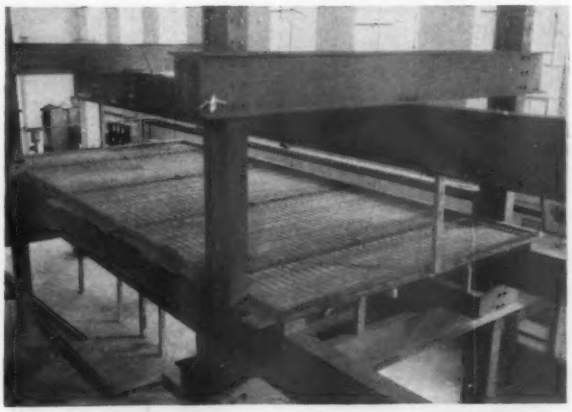

Fig. 1.—Specimen during construction

(b) *Loading Arrangement and Instrumentation*

The span of the steel joists was kept constant throughout the test series. Likewise, the strength and thickness of the concrete slabs were kept as constant as possible so that only the spacing of the joists, and thus the flange width of the effective tee-beam was allowed to vary.

Table 1.—Test Specimens Details

	SPECIMEN No.			
	1	2	3	4
Span of I-Beams (4 in. × 3 in. × 10 lb. BSB)	9 ft. 6 in.	9 ft. 6 in.	9 ft. 6 in.	9 ft. 6 in.
Spacing of I-Beams (4 in. × 3 in. × 10 lb. BSB)	2 ft. 6 in.	5 ft. 4 in.	6 ft. 8 in.	8 ft. 0 in.
Slab Thickness	2 in.	2 in.	2 in.	2 in.
28 days slab strength lb/sq. in.	5400	4580	3450	4990
Mortar Mix For Slab	1 part of cement : 3 parts of crushed granite stone fines passing 3/16 in. sieves, by weight. Water Cement Ratio = 0·55			
Shear Connectors on the Top Flange of I-Beams	Double spiral of S.W.G. No. 7 at 3 in. pitch with 1 in. coil diam.			
Modular Ratio $m = E_s/E_c$	9·81	10·93	13·28	10·82

All specimens were tested under equal concentrated loads applied through steel blocks direct to the top flange of each joist at the mid-span points. Loading was applied by means of hydraulic jacks through ball seatings and spreader beams and was carried out in incremental stages up to a maximum load of three tons per joist. Fig. 2 shows the loading arrangement, and the steel blocks through which the loads were applied to the joist flanges can be seen in Fig. 1.

Strains at the top and bottom surfaces of the slab were measured with ' Demec ' 8 in. tensometer gauges while Tinsley electric resistance gauges were used to determine those at the top and bottom flanges of the steel joists. All strains were measured parallel to the span of the beams.

Vertical deflexions of the joists were measured by means of dial gauges reading to 0·001 in. Dial gauges were also used to measure the relative movement between the slab and steel joists, commonly referred to as the slip.

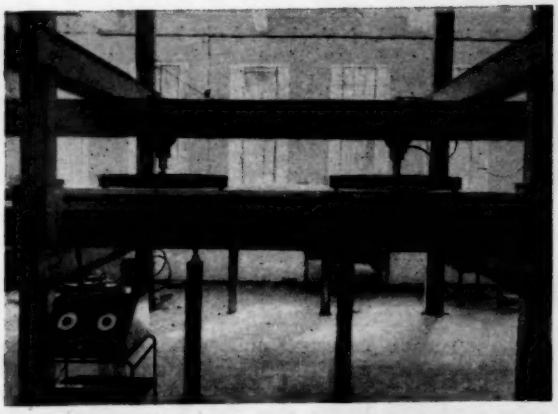

Fig. 2.—Loading arrangement

Experimental Results

(a) *Transverse Distribution of Longitudinal Strains at Slab Surface*

In order to detect the shear lag effect and hence to determine the effective width of the composite tee-beam flange, longitudinal strains were measured on the top and bottom slab surfaces at different sections across the slab (i.e. at right angles to the longitudinal beam axis). Figs. 3, 4, 5 and 6 show the measured strain distribution across the top slab surface for all four specimens for sections $L/8$, $L/4$, and $3L/8$ from one support where L is the span of the beam.

Since in elastic design the effective width of a tee-beam flange is determined by the maximum stress developed in the uppermost slab fibre, the strain distribution curves shown in Figs. 3, 4, 5 and 6 have been used to obtain experimental values of effective width of tee-beam flange by summing up the area under the curve for a tee-beam by graphical integration and dividing by the maximum strain ordinate in that area. The values thus obtained are shown in Table 2 under the heading, Method I.

(b) *Strain Distribution in The Composite Tee-beam.*

(i) Distribution of Strains in Plane of Web at Beam Cross-Sections.

Fig. 7 shows the strain distribution down the plane of the joist web for all four specimens at sections $L/8$, $L/4$ and $3L/8$ measured from one support. As can be seen from the location of the strain gauges shown on the beam cross-section in Fig. 7, the recorded strains were not all measured in the plane of the joist web. Furthermore, the concrete strains represent the mean values over an 8 in. gauge length whereas the steel strains are measured over a joist length of approximately $\frac{3}{4}$ in.

Examination of the strain distribution diagrams of Fig. 7 shows discrepancies between the measured concrete and steel strains at the contact faces of the two materials. Such discrepancies are not unusual in composite beam tests and are attributable primarily to slip between the concrete and steel along the contact surface, and to a lesser extent to instrumental errors arising from eccentricities of gauge positions and differences of gauge lengths as mentioned above.

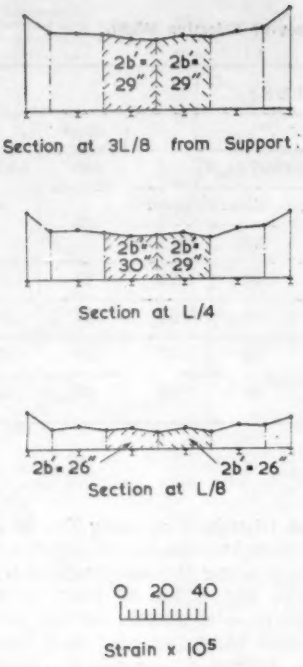

Fig. 3.—Transverse distribution of longitudinal strains. Specimen No. 1.—3-ton load at each beam midspan. Beam spacing/beam span = 30 in./114 in. = 0·26

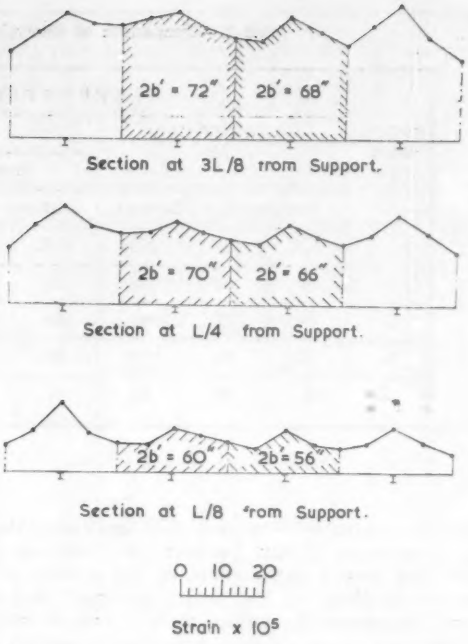

Fig. 5.—Transverse distribution of longitudinal strains. Specimen No. 3.—3-ton load at each beam midspan. Beam spacing/beam span = 80 in./114 in = 0·70

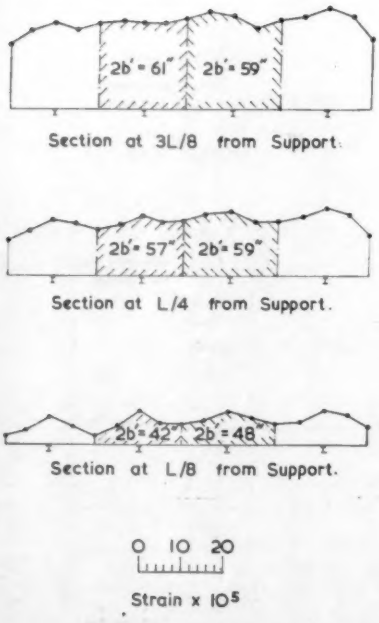

Fig. 4.—Transverse distribution of longitudinal strains. Specimen No. 2.—3-ton load at each beam midspan. Beam spacing/beam span = 64 in./114 in. = 0·56

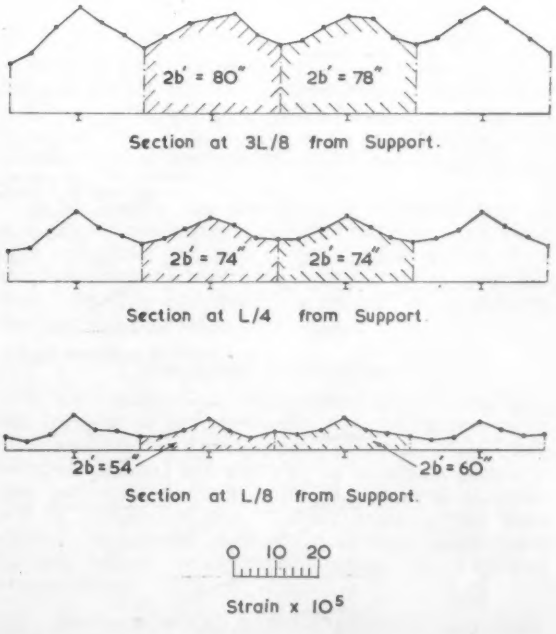

Fig. 6.—Transverse distribution of longitudinal strains. Specimen No. 4.—3-ton load at each beam midspan. Beam spacing/beam span = 96 in./114 in. = 0·843

Table 2.—Comparison of Analytical and Experimental Values of Effective Width

SPECIMEN No. (1)	EFFECTIVE WIDTH (INCHES)										
	ANALYTICAL (2)			EXPERIMENTAL						Steel Designers' Manual (5)	AASHO (6)
				Method I (3)			Method II (4)				
	Distance From Support			Distance From Support			Distance From Support				
	L/8	*L*/4	3*L*/8	*L*/8	*L*/4	3*L*/8	*L*/8	*L*/4	3*L*/8		
1	30	30	28	26	30	29	22	24	26	27	24
2	54	52	45	45	58	60	37	48	54	27	24
3	57	54	48	58	68	70	40	52	57	27	24
4	63	59	51	57	74	79	40	40	42	27	24

Earlier researches[7,8] in this field indicated that a small amount of slip between the concrete and steel had only a slight effect on the strains at the outermost fibres of the beam, although such slip would influence the strains at the contact surfaces of the two materials to a considerable extent. If, on the basis of these findings, the measured strains at the top surface of the slab and the bottom surface of the steel joist are assumed as reasonably reliable, the neutral axis may be located at the point of zero strain obtained by plotting those strains and joining by a straight line as shown in Fig. 7. When the position of the neutral axis is known the effective width of the slab at the tee-beam flange can be determined in the usual manner. The effective flange widths obtained in this manner are given under Method II of Table 2.

(ii) Strain Distribution Along The Beam Span. The strain distributions at the upper slab and lower beam surfaces along the longitudinal beam sections are shown in Fig. 8 for all four specimens. The plotted experimental strains are the average values for all interior beams in each test specimen. For comparison purposes, computed strains are also shown on the same figure. These were obtained by assuming the effective width of compression flange equal to (a) the full width of beam spacing, and (b) the value recommended in the Steel Designers' Manual[5], i.e. the least of the following :

(1) one third of the effective span of the tee-beams;
(2) the distance between centres of the ribs of the tee-beams;
(3) the breadth of the rib plus twelve times the slab thickness.

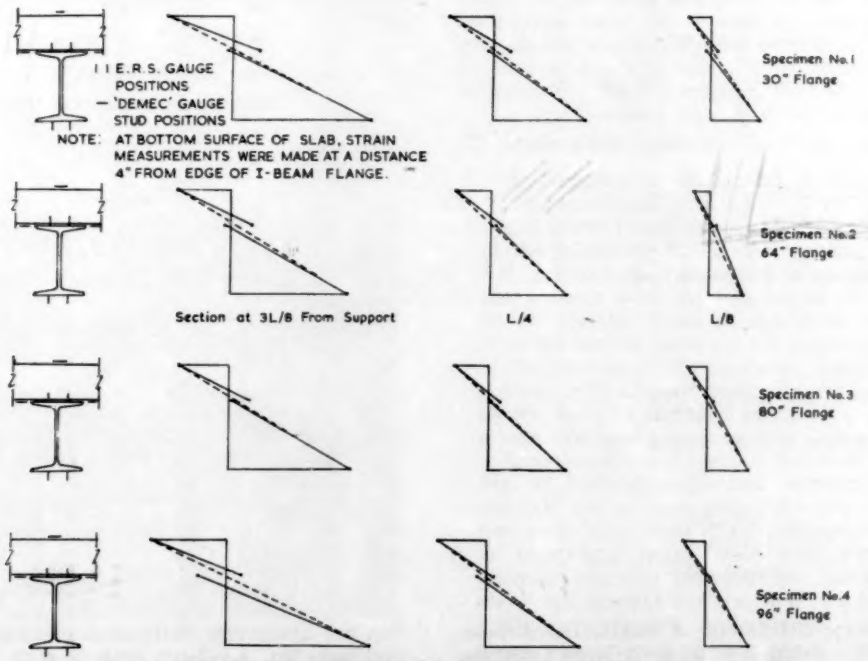
I I E.R.S. GAUGE POSITIONS
— 'DEMEC' GAUGE STUD POSITIONS
NOTE: AT BOTTOM SURFACE OF SLAB, STRAIN MEASUREMENTS WERE MADE AT A DISTANCE 4" FROM EDGE OF I-BEAM FLANGE.
Specimen No.1 30" Flange
Specimen No.2 64" Flange
Specimen No.3 80" Flange
Specimen No.4 96" Flange
Section at 3L/8 From Support
L/4
L/8

Fig. 7.—Distribution of strains in plane of web at beam cross-sections

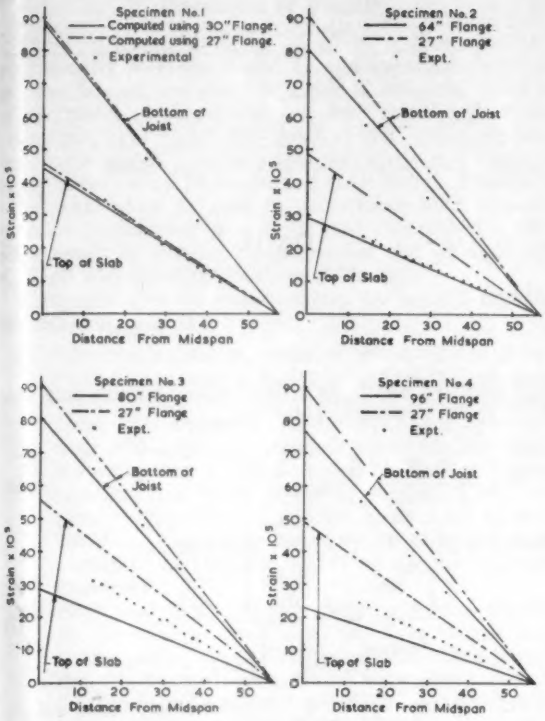

Fig. 8.—Distribution of strain along beam section.

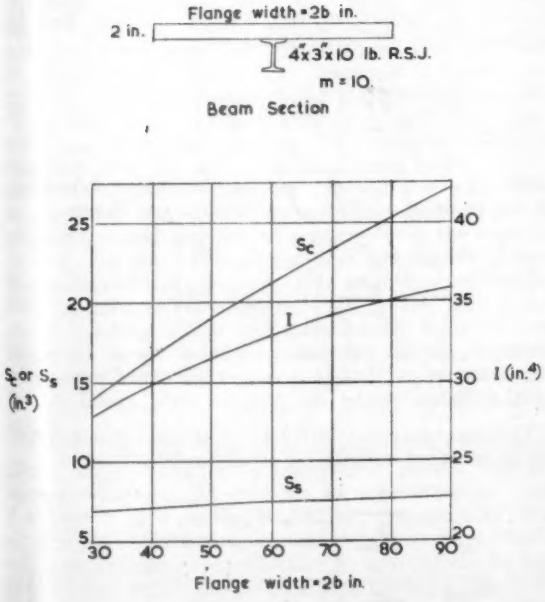

Fig. 9.—Effect of flange width on properties of composite beam

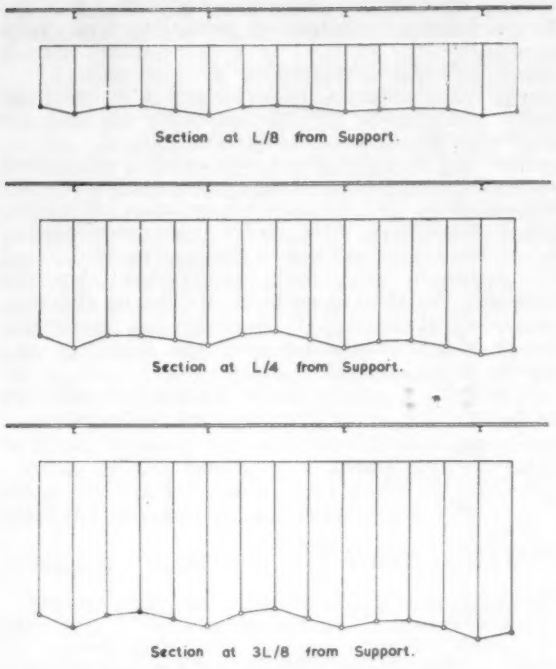

Fig. 10.—Measured deflexions—Specimen No. 3.

It is clear from Fig. 8 that the strains at the bottom of the joists are not greatly affected by the width of the tee-beam flange. This fact is further illustrated in Fig. 9 which shows that the influence of flange width on the value of S_s, the sectional modulus of the composite section with respect to the steel at the bottom surface, and hence on the strains in the steel, is small.

(c) *Deflexion Measurements*

Vertical deflexions were measured at various beam sections for all specimens. In addition, intermediate deflexion readings were taken under the slab of Specimen No. 3 at selected sections between the beams. Typical results of these observations are shown in Fig. 10.

Under loading, the slab surfaces assumed a series of dish-shaped profiles in which the maximum deflexions occurred under the loading points, and decreased as the mid-points between the beams were approached, thus indicating that both longitudinal and transverse bending was present in the slabs.

Analytical Investigation

Various mathematical approaches[9,10,11,12] have been put forward to determine the effective flange width of a tee-beam of homogeneous material, which may form an isolated beam having a wide thin flange or may be one of a series, as in the case of a plate, stiffened with a number of ribs at right angle to the plate surface. In general, most of the approaches are based on the plane stress theory using the following assumptions :

(1) thickness of slab is small compared with depth of beam
(2) transverse bending of flange is neglected,
(3) during bending of the beam, forces are transmitted to the flange at its middle plane.

Thus, the solution of the stress distribution in the flange is reduced to integration of the differential equation :

$$\frac{\partial^4 F}{\partial x^4} + 2\frac{\partial^4 F}{\partial x^2 \partial y^2} + \frac{\partial^4 F}{\partial y^4} = 0 \quad \cdots \quad (1)$$

where F is the Airy stress function.

The validity of the above approaches when applied to composite tee-beam design, however, has not yet been studied. At this preliminary stage of investigation, and in view of the limited experimental results, it appears logical to examine the existing theory and to compare its values with the experimental results obtained. No effort is made to establish an elaborate mathematical solution to suit the test specimens, and in the discussion following, the theory is substantially that given by Schade[9].

Taking the x- and y-axes for the multi-rib structure as shown in Fig. 11 the stress function of the form :—

$$F = \sum_{n=1}^{\infty} (f_n) \sin \frac{n\pi x}{L} \quad \cdots \quad (2)$$

where $(f_n) = A_n \cosh \frac{n\pi y}{L} + B_n \sinh \frac{n\pi y}{L}$

$$+ C_n \frac{n\pi y}{L} \cosh \frac{n\pi y}{L} + D_n \frac{n\pi y}{L} \sinh \frac{n\pi y}{L} \quad \cdots \quad (3)$$

satisfies equation (1). The subscript n is used since the function (f_n) differs for each value of n.

The four constants A_n, B_n, C_n, and D_n are determined from the known boundary conditions, which for the particular case considered, are assumed as follows.

(1) Since the x-axis is an axis of symmetry, there can be no transverse displacement along this axis, therefore, at $y = 0$, $v = 0$ for all values of x. Thus,

$$\text{at} \quad y = 0 ; \qquad \frac{\partial v}{\partial x} = 0.$$

(2) No transverse displacement at beam sections, i.e.

$$\text{at} \quad y = b ; \qquad \frac{\partial v}{\partial x} = 0.$$

(3) The shearing stress along the x-axis by symmetry is zero, i.e.

$$\text{at} \quad y = 0 ; \qquad \frac{\partial \tau xy}{\partial x} = 0.$$

The above boundary conditions give

$$B_n = C_n = 0 \quad \text{and}$$

$$D_n = -\frac{A_n \tanh \frac{n\pi b}{L}}{\frac{n\pi b}{L} - \left[\frac{1-u}{1+u}\right] \tanh \frac{n\pi b}{L}} \quad \cdots \quad (4)$$

where u = Poisson's ratio.

It is noted that the function F cannot satisfy completely the conditions of a plate with free ends, because it results in a set of horizontal shearing stresses along the edges i.e. at $x = 0$ and L. The effect of this was explained by Schade[9] and also by Winter[11].

The Effective Width

A representative distribution curve for the longitudinal stress, δ, in the flange plate is shown in Fig. 11. This stress varies across the width of the plate and reaches maximum at rib intersections. The force in the flange of unit thickness is represented by the area under the curve and can be expressed as

$$P_n = 2\int_0^b \delta \, dy \quad \cdots \quad (5)$$

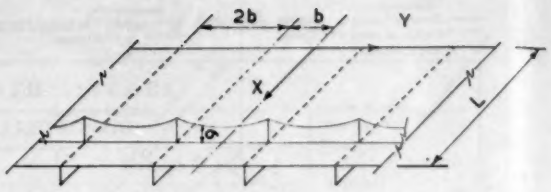

Fig. 11.

The effective width, $2b'_n$, is obtained by dividing this force by an optimum stress, δ_{max} i.e.

$$2b'_n = \frac{P_n}{\delta_{max}} = \frac{2\int_0^b \delta \, dy}{\delta_{max}} \quad \cdots \quad (6)$$

since δ is given by $\partial^2 F / \partial y^2$, therefore,

$$\frac{\int_0^b \delta dy}{\delta_{max}} = \frac{\left.\frac{\partial (f_n)}{\partial y}\right]_0^b}{\left.\frac{\partial^2 (f_n)}{\partial y^2}\right]_{y=b}} \quad \cdots \quad (7)$$

Differentiating equation (3) accordingly, and substituting into equation (7) the following is obtained.

$$\frac{b'_n}{b} = \frac{4}{\alpha} \frac{\cosh \alpha - 1}{(3+u)\sinh \alpha - \alpha(1+u)} \quad \cdots \quad (8)$$

where

$$\alpha = \frac{2n\pi b}{L}$$

Values obtained from equation (8) are for beams under a sinusoidal loading.

When the applied load is not sinusoidal, the bending moment M applied at each stiffener section may be represented by a Fourier series :—

$$M = \sum_{n=1}^{\infty} \left[M_n \sin \frac{n\pi x}{L} \right] \quad \cdots \quad (9)$$

where M_n is the coefficient of the series depending on the value of n. Then, by simple beam theory,

$$\delta_{max} = \frac{M}{S_n} = \sum_{n=1}^{\infty} \left[\frac{M_n \sin \frac{n\pi x}{L}}{S_n} \right] \quad \cdots \quad (10)$$

where S_n is the sectional modulus for a tee-beam consisting of a rib and a width of the flange plate equal to $2b'_n$.

The final effective width for the actual applied bending moment represented by equation (9) is then

$$2b' = \frac{P}{\delta_{max}} = \frac{\sum_{n=1}^{\infty} \left[\frac{2b'_n M_n \sin \frac{n\pi x}{L}}{S_n} \right]}{\sum_{n=1}^{\infty} \left[\frac{M_n}{S_n} \sin \frac{n\pi x}{L} \right]} \quad \cdots \quad (11)$$

Discussion of Results

The experimental values of effective flange width, obtained by Methods I and II and averaged over the interior beams, are given in Table 2, columns 3 and 4 respectively. Analytical values calculated from equation (11), using the first eleven terms of the bending moment Fourier series, are given in column 2 of the same table. In computing these values, Poisson's ratio was taken as zero in accordance with normal practice. Columns 5 and 6 give respectively, the corresponding values recommended for design by AASHO and the Steel Designers' Manual.

Comparison of the values shown in Table 2 reveals the following points :—

(1) Whereas values of effective width obtained by analysis are least at midspan section and increase towards supports, those obtained by experiment have the opposite tendency, being greatest near mid-span and decreasing towards supports.

(2) Experimental values obtained by Method II are smaller than those given by Method I and in closer agreement with the analytical values. However they differ from the latter in so far as variation of effective width along the span is concerned.

(3) Both analytical and experimental values, except in the case of Specimen No. 1 in which the beam spacing is relatively small, are much larger than the values recommended for design purposes as shown in Columns 5 and 6.

There are several reasons which, either individually or collectively, may account for the differences between theory and practice outlined in (1) above. On the theoretical side, the basic assumptions upon which the fundamental theory is based are far from being realised in practical composite beam construction. In the specimens tested the slab could not be considered as thin compared with the depth of the steel joists, and furthermore, noticeable transverse bending developed in the slabs as indicated in Fig. 10.

Equation (11) gives values of effective flange width which for a given set of conditions, may vary along the span according to the nature of the Fourier series used to represent the applied bending moment. For a single concentrated load, the effective width is least at the loaded section and increases towards the supports ; for uniformly distributed load, it is maximum at midspan and decreases slightly towards the ends, while for sinusoidal loading, the effective width is constant over the span. Whether this holds true or not for an actual structure has not yet been fully investigated and the Authors doubt that an actual structure would respond so sensitively to the type of loading. In any event, theoretical refinements of this nature have little practical value at the present stage, since membrane stresses, set up in the slab itself under bending action will undoubtedly influence the effective flange width under the higher loading intensities, and such stresses cannot be accounted for in the linear plate theory, on which equation (11) is based.

Apart from the theoretical considerations discussed above, slight errors in reading the 'Demec' tensometers, which might give strain differences of 1 or 2×10^{-5} if not carefully handled, or non-uniformity of the slab thickness, could result in appreciable errors in estimation of effective flange width. In the present test series special care was taken to minimize errors from such causes as far as possible. The divergence between theory and practice which has occurred in so far as variation of effective width is concerned, has been consistent throughout the tests which in itself, tends to confirm the reliability of the test results.

The differences in experimental effective flange width values mentioned in (2) are undoubtedly due to the different techniques adopted to determine those widths. In Method I the effective widths have been determined from the extreme fibre strain distribution across the flanges measured at the upper slab surface which takes into account the effect of transverse bending of the slab. Method II, on the other hand, uses the strain distribution on a vertical plane through the web of the beam section and hence excludes the effects of transverse bending except in so far as such bending may influence the concrete strains directly over the beam web. In this way it lies closer to assumption (2) of the analysis, which may account for the better agreement obtained with the theoretical values than has been given by Method I. It should be noted, however, that in Method II, an error in the position of the neutral axis arising from possible errors in strain measurements would significantly affect the resulting values of flange width.

Relation of b'/b and $2b/L$.

The variation of b'/b against $2b/L$ can be examined from Fig. 12 from which it can be seen that both the experimental and analytical results show the same tendency, i.e. values of b'/b decrease with increase in the ratio $2b/L$. The rate of decrease of the analytical values, however, is more rapid. Consequently analytical values of effective width are smaller than the experimental ones. Based on the present test results, it can be reliably stated even at this preliminary stage,

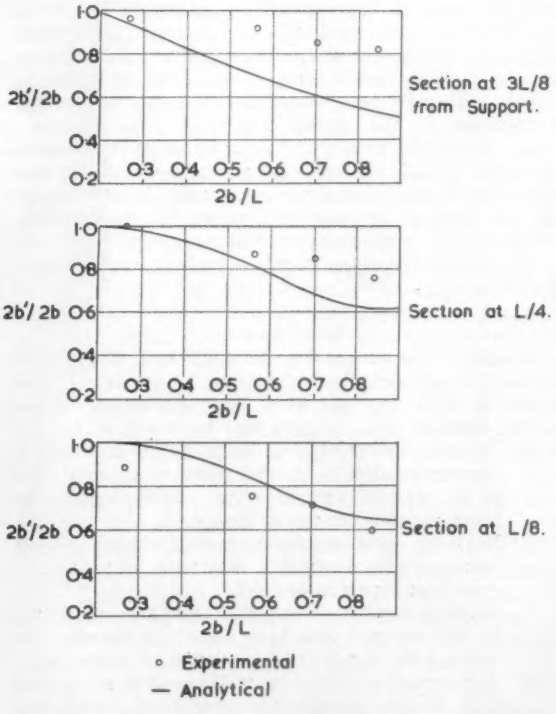

Fig. 12.—Variation of effective flange with the ratio $\frac{\text{Beam spacing}}{\text{Beam span}}$

Table 3.—Analytical Effective Widths

SPECIMEN NO.	EFFECTIVE WIDTH	
	By Equation (11) at $3L/8$ From Support	Equation (8)
1	28	27
2	45	43
3	48	45
4	51	48

that the effective width of a tee-beam compression flange varies with the ratio $2b/L$ and that analytical values obtained from equation (11) can be used in design with a fairly large margin of safety.

It is fully realised that application of equation (11) for design purposes is tedious, since it is necessary to compute a new sectional modulus S_n for each term of the series. But if only the first term, instead of the first 11 terms, of the series is used to calculate the effective width, the amount of work involved is much reduced. Table 3 gives a comparison of the values of effective width obtained by equation (11) using the first 11 terms of the series and by equation (8) i.e. using the first term of the series ($n = 1$). The differences are not excessive, and for design purpose, equation (8) can be used with advantage.

Effect of Slab Thickness.

The effect of slab thickness on the effective width has not been studied experimentally. According to the analysis, the effective width is a function of the bending moment represented by a Fourier series as well as the sectional modulus S_n. Therefore, for a single harmonic, effective width is independent of thickness. It is only when the values of sectional modulus S_n corresponding to each harmonic are computed for the summation of the series that the slab thickness, which affects the values of S_n comes into play. Hence the effect of slab thickness on effective width depends upon the loading condition, and may be significant for one type of loading and negligible for another. However, the applicability of this analytical deduction to an actual structure requires further experimental investigation.

Conclusion

Although experimental results are still too meagre to permit any definite conclusions to be drawn, the following observations, from comparison of experimental and analytical results may be noted.

(1) Effective width of a tee-beam flange determined experimentally is larger than that predicted by theory and is much larger than that currently recommended for use in design.
(2) Both the test results and analysis agree that values of effective width vary with beam spacing over beam span ratio ($2b/L$).
(3) Loading conditions and slab thickness, as shown by the analysis, also have effects on the effective width, but experimental confirmation is necessary.
(4) Experimental effective widths vary along the span, but in an opposite sense to that obtained by analysis.

With further studies both on the experimental and theoretical side, values of effective width for a composite tee-beam flange may be predicted with reasonable accuracy and, consequently, more economical design can be achieved.

Acknowledgement

The work described in the paper was carried out in the Peel Engineering Laboratory of the Department of Civil Engineering, University of Hong Kong. The Authors are grateful to the Research Grants Committee of the University of Hong Kong for financial help which permitted this investigation to be undertaken.

Thanks are also due to Mr. F. K. Kong, a post-graduate student, who was intimately concerned with the investigation, and to Mr. E. R. Sutton and the staff of the Ho Tung Workshop for help received during testing.

Notation

A_n, B_n, C_n, D_n = Constants corresponding to the nth harmonic.
$2b$ = Distance between centre to centre of floor beams.
$2b'$ = Effective width of tee-beam flange.
$2b'_n$ = Effective width of tee-beam flange corresponding to the nth harmonic.
F = Airy stress function.
f_n = Function of y corresponding to the nth harmonic.
L = Span of beam.
M = Bending moment.
M_n = Bending moment corresponding to the nth harmonic.
n = Subscript, used to represent the nth harmonic.
P = Force in the tee-beam flange of unit thickness.
S_n = Sectional modulus of a tee-beam consisting of a rib and a width of the flange equal to $2b'_n$.
S_s = Sectional modulus of a composite beam with respect to steel at bottom surface.
u = Poisson's ratio.
v = Displacement in y direction, or transverse displacement.
x = Distance measured from support of beam along the direction of the x-axis (longitudinal direction).
y = Distance measured from x-axis along y-direction (transverse direction).
τ_{xy} = Shearing stress.
δ = Stress in x-direction, longitudinal stress.
α = $2n\pi b/L$.

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Discussion

The Council would be glad to consider the publication of correspondence in connection with the above paper. Communications on this subject intended for publication should be forwarded to reach the Institution by December 31st, 1961.

Book Reviews

Béton Précontraint (Étude Théorique et Expérimentale), by Y. Guyon, Tome II (Constructions Hyperstatiques): (Paris: Editions Eyrolles, 1958). 9½ in × 6 in., 818 pp., many illustrations and tables.

Also the English Translation:

Prestressed Concrete, Vol. II. (Statically Indeterminate Structures). 9½ in. × 6 in., 741 pp. Numerous illustrations and many tables. (London: Contractors' Record; New York: Wiley, 1960). 95s.

The French edition has been very well translated by Mr. C. van Amerongen, who added valuable footnotes to clarify some points. The author has gained great international reputation by his work devoted to prestressed concrete. He was the first—as early as 1945—to introduce its application to statically indeterminate structures (which present most complex problems) and since then he has published numerous comprehensive papers on this important and difficult subject.

This second volume of M. Guyon contains all these studies in 19 chapters (Nos. XIX to XXXVII, each containing between 4 and 21 sections) and has therefore become very comprehensive and somewhat unwieldy. In these circumstances there is a certain amount of repetition and lack of uniformity, which is understandable since it contains all the developments of the author's studies and research on the subject.

This standard work certainly contains a volume of knowledge and will therefore be undoubtedly of greatest interest to the specialist engineer, particularly the research worker. It is limited to statically indeterminate structures of almost constant cross section subjected mainly to bending. Thus shell roofs, domes, arches and other such structures are excluded.

M. Guyon is basically a great mathematician and has investigated the very important question of dangerous cracking, plastic hinges and failure on statistical considerations obtained from all the tests available. He states in his preface: "Owing to the lack of a sufficient number of data capable of statistical evaluation it is often necessary to have recourse to reasoning in order to establish connections between certain observed results. It will in the future therefore be necessary to reconsider, or to state more precisely, quite a number of the conclusions given in the present treatment of the problem..." Thus some of his most ingenious theories are in fact only conjectures, as the author says himself. It certainly will be very interesting to find out how they agree with future exact tests.

M. Guyon is most thorough and exact in his investigations, for example, he shows that a two-pinned rectangular frame must be considered as a twelve times statically indeterminate structure, but he indicates how the solution can be simplified by trial and error.

Particular attention has been paid by the author to the question of the delayed visibility of cracking in statically indeterminate structures in view of stress redistribution, and he has come to the following conclusions: "any engineer will consider a structure satisfactory if a rigorous visual inspection fails to reveal any apparent cracking" and "there is a threshold of visibility below which we pronounce the concrete to be uncracked." These are most important conclusions by the author who has included in this book many valuable new findings relating to the simply supported beam which was dealt with ten years earlier in the first volume.

This comprehensive volume thus contains a wealth of knowledge and all concerned, author, translator and publishers, must be congratulated on this important standard work. However, it is most regrettable that an alphabetical index is missing. Any reference is limited to the heading of chapters and sections, given in the summary of contents at the beginning of the book; thus it is most cumbersome for the reader to find a required place after a long search. It is stated in the translator's foreword that the book "calls for patient study, and the subjects dealt with are so intimately interrelated as to make a brief 'looking-up' of individual items almost pointless." It is correct that it is not easy to prepare a simple alphabetical index to such a voluminous standard work, which is not a textbook of uniform character, but the result of almost 15 years' study and development, with the consequence that the subjects are intimately interwoven. Nevertheless, it is hoped that the Publishers will be prepared at the next edition to have such an index prepared and thus make this most valuable book more suitable for practical use.

P.W.A.

The 16-in. E.R.W. Tube Plant, Shotton *

Discussion on the Paper by W. T. Brooks, T. Burnett-Stuart (Graduate) and G. Bernard Godfrey, A.M.I.Struct.E., A.M.I.C.E., A.M.I.Mun.E.

Mr. O. BONDY (Member), described the paper as impressive, and invited the authors to enlarge on one or two technical points.

It had been most interesting, he said, to listen to a description and see the illustrations of a remarkable structure, by those who were responsible for its design, fabrication and erection. That feeling was enhanced by the fact that some features of the structure—and he referred particularly to the steelwork part of it—seemed to be somewhat unorthodox, perhaps even novel, in certain aspects.

In the first paragraph of the Introduction to the paper it was stated that the new plant would produce welded oil-line pipes up to 16 in. outside diameter. He asked if one should understand that they were not to be used for structural purposes. As a sub-question, would it be all right to use E.R.W. tubes formed into hollow rectangular sections?

With regard to the electric overhead travelling cranes, it was obvious that wherever there were moving parts of a structure, there was a strong case for considering tubular members; and the crane makers' reluctance to consider improved types of design, such as shown in Fig. 11, was not easy to understand.

On numerous occasions he had used tubes in his designs, and he was particularly interested in the horizontal surge girders for crane gantries as illustrated in Fig. 12. Tubes were the ideal material for the diagonals and posts which have to withstand buckling due to surge in either direction.

The tubes he had used in one such case were of 3½ in. outside diameter and 7 gauge thick; the price per ton at that time was nearly twice that of rolled steel sections, but even so it was more economical to use tubes.

Asking what sort of inspection and testing was used, especially for the site-welded joints in the 90 ft. crane girders, Mr Bondy said he hoped no extravagant application of site testing methods had to be paid for, as was the case in connexion with one or two very large tubular structures in recent years.

Commenting on the unit selling rates given in Table 2, which included the rates per ton for tubular crane girders, plate girders and compound girders, he said the comparison was most interesting. He could not quite understand whether the tubular crane girders had been constructed using tubes only, nothing else; he had not seen them illustrated in the authors' slides.

Having noted that there was only a tiny difference between the rates of £93 per ton for tubular crane girders and £91 15s. 0d. per ton for plate girders, he asked if it could be regarded as quite reasonable for designers to put forward tubular designs without worrying too much about the cost. So far in his limited experience the cost aspect had nearly always been the decisive factor. If in the open market the price for tubes could really compete against that of rolled steel sections then he felt sure that many designers would be delighted to adopt tubes.

In conclusion, he said he had found the paper most useful and enjoyable to read; he offered his congratulations to the authors for having given so much attention to the details of design; such effort with somewhat novel types of construction could well decide their future prospects.

Dr. J. S. TERRINGTON (Associate-Member), said he was very much impressed by the amount of detail given in the paper; the authors were to be congratulated on that, including the cost and weight figures in Table 2, on the paper as a whole, and, indeed, on the structure itself.

After the spectacular saving in weight achieved on the long span at Haslar, he had looked for corresponding amounts here or, failing this, paramount advantages by the use of tubes.

With regard to the gantry girders, he said that people who had been following developments in welding had been aware that some defects due to fatigue had arisen in girders which had been in commission for some time, and he asked if the authors had considered fatigue in the course of their design. Mr. Bondy had mentioned that the surge girders were particularly well suited to tubular construction. On the other hand, it was just in that part of the girders that fatigue defects had occurred and he wondered if the authors felt confident that ten years hence they would still be giving good service.

North light construction was well known to structural engineers, of course. But, apart from the purely architectural point of view, on one or two occasions at conferences he had heard the view expressed that a terribly cold light existed in buildings of that type, and in fact he had noticed it himself, whereas when the lighting was from all directions there was a brighter psychological atmosphere. However, that was perhaps a secondary consideration; the paper was concerned with the economics of the structure.

Mr. T. BEDFORD (Member), added his congratulations to the authors for two reasons, first for the high merit of the paper and the very interesting information on costs, and secondly, on being the successful contractors when tenders had been obtained from seventeen civil engineering firms and six steelwork firms. They could only have succeeded either by their great ingenuity in design or by quoting low prices. At the same time he expressed the view that to ask for seventeen tenders was rather excessive.

Concerning the ground work he had already asked Mr. Godfrey about the piles and was assured that the working load on the 14 in. square pile, using a load factor of 2·5, was 124 tons on this site. He had queried that because he had thought it was a very high figure. It was also stated that 12 in. piles were actually used which carried a maximum load of 41·5 tons. Could the authors say why the 12 in. piles were used and why they were limited to 41·5 tons when a 14 in. square pile was capable of carrying a working load of 124 tons.

The level of the site was raised by about 8 ft. by filling with sand and the floor slab was placed on

* *Read before the Institution of Structural Engineers at* 11, *Upper Belgrave Street, London, S.W.*1, *on the* 10*th November*, 1960. *Lt.-Col. G. W. Kirkland, M.B.E.(Mil.), M.I.Struct.E., M.I.C.E., (President) in the chair. Published in "The Structural Engineer," Vol.XXX VIII, No.* 11, *pp.* 334-345.

the filling, and he asked what was the safe load carrying capacity of the sand and what was the imposed load on the floor slab.

As to the structure itself, he asked why the north light design was used. He personally could not see much justification for a north light roof unless the manufacturing processes to be carried on in the building demanded that sunlight should be excluded; and in a steel mill one could not think that some sunshine would be detrimental. It might do the employees a lot of good and cheer them up. But the main point was that a north light roof must be something like 25 per cent. more expensive than the span type of roof, and one wondered why the designers did not provide for an 80 ft. span roof with an adequate amount of glass.

The chopping off of the bottom booms of the crane girders had been commented on by Mr. Godfrey. It was unusual and it meant that there was loss of continuity; one wondered why, in a welded or mainly welded structure, the designers did not take advantage of continuity. He also asked whether any vibrations had occurred in the girders due to the fact that by disconnecting the bottom boom the torsional rigidity and stiffness of the girders was obviously reduced.

On the question of costs, he said the tenders seemed to be very competitive and the cost of construction of the building at 25/- per square ft., including the covering, as shown in Table 2, was remarkably low for that type of building.

As to the tubular content of the job, one noted that the framework of tubes weighed about 720 tons, and it seemed to work out that the cost of the tubular steelwork and its erection was £129 per ton. He had found that by using tubular structures one expected to save about 33⅓ per cent. in weight as compared with rolled steel construction. That meant that, if the tubular steelwork cost £129 per ton, by comparison rolled steel sections should be £86 per ton. Those who had not had so much experience of tubular steel but did know rolled steel prices would judge that £86 per ton for this job would be a very low figure, and Mr. Bedford doubted very much that it could be done at that price.

The figures showed that the whole structure had been very well designed and at very low cost.

Mr. CHARLES H. A. BOYSE (Messrs. Stewarts and Lloyds, Limited), opened his remarks by saying that he wished to endorse the congratulations already made by a number of speakers to the authors on their preparation of the paper.

Raising some additional points on which he would be interested to hear further comments, he said that he had heard it suggested that the type of design adopted for the tubular structure of the Shotton building occasioned additional moments to be exerted on the foundations. He did not know if this suggestion were fact, and would like to know from the speakers if the civil engineering costs were affected by this factor.

Brief reference had been made to the relative economics of R.S.J. and tubular sections for columns. All things being equal, it would seem sound, both practically and aesthetically, in a mainly tubular structure to use tubular columns, but if there were a significant cost disadvantage, perhaps the speakers would care to expand on the point. They also might care to comment on the economics of the use of R.H.S. members in either roof or column structure.

Following up the remarks by Mr. Bedford and Dr. Terrington concerning the type of roof structure that was chosen, Mr. Boyse said that he himself was responsible, to a great extent, for its choice, so would like to justify it. There were two main reasons for the choice, the first of which was personal comfort. Even under English summer conditions the normal type of glazing installed in a pitched roof could, at times, produce unpleasantly hot working conditions and very strong lighting contrasts. The second point was that in an industrial area where heavy air pollution existed, the problem of cleaning glass was a serious one. The vertical glass in a north light roof reduces deposition of dirt to a minor problem. Where also a walkable gutter is installed at the foot of the glazing such infrequent cleaning as is required is a simple operation as the whole glazed area is within the reach of a man.

The ultimate consideration in regard to roof construction, as with other parts of a structure, was, of course, that of price. As had already been noted by another speaker in the discussion, the overall cost of the building was very reasonable, therefore it could be concluded that the north light roof was not an expensive luxury with this type of construction.

On the point that seventeen firms were invited to tender in respect of foundations and civil engineering work, Mr. Boyse agreed that it would be a reprehensible practice if all those firms were asked for full tenders. This, in fact, was not the case, only target figures and unit rates being requested.

Finally, on Mr. Bondy's reference to the use of welded tubes—the products of this plant—for structural purposes, he assured the meeting that the sections were perfectly suitable for that purpose, possibly more so than other types of tube, but he wished to invite the authors to enlarge on this point.

Mr. S. B. TIETZ (Associate-Member), recalled that some years ago a preliminary design was prepared for a job in which the ideal would have been the use of tube in conjunction with R.S. sections, but it was almost impossible then to obtain a tender for a design incorporating both tubes and other rolled sections. He asked if there were still a disinclination to mix them and, if so, why.

Mr. F. R. BULLEN, (Vice-President) said the paper was interesting and it had been made abundantly more interesting by the additional information that had been given at the meeting; he had found some of the authors' introductory remarks to be particularly valuable.

On the matter of cost, he asked how the final figures compared with those in the lowest tender, and whether there was in fact any relationship between the original tender figure and the final cost. He had also wondered whether the cost of the foundation work per square foot could be included.

At the end of the paper the authors referred to the concrete being given two coats of what was called 'Lithurin' at least 14 days after the concrete was laid. Mr. Bullen wondered whether they really meant 'at most' 14 days. He had also wondered how the concrete was put down, what sort of concrete it was, what tamping was used and whether vibration was applied. Further, was the top coating laid as an additional operation and—with great respect—had it become loose since it was finished?

Again, what did they do about drains, conduits or trenches and such inevitable things as came into all these jobs?

On the question of piling, he recalled Mr. Bondy's reference to another job in which tubular steel was used, and said that on that same site he had calculated that the friction on the piles was about 17 cwts. per

sq. ft.; if one used something like that figure for the piles at Shotton, one arrived at an ultimate value of 87 tons for the 14 in. × 14 in. pile. He had tried to make use of the figures given for that test pile which was subjected to ten blows of a three-ton hammer; if the authors could give details of the helmet packing it would help. He had calculated a figure of 247 tons on the toe of that pile to which the friction should be added, making a total of 334 tons.

Mr. GODFREY said that it was in fact 308 tons.

Mr. BULLEN asked if that were calculated on the basis of the revised Hiley formula, because if it were he would like to ask what was the revised Hiley formula! If indeed it was anything like 247 tons on the toe of the 14 in. × 14 in. pile (not in friction) it came to about 180 tons/sq. ft. That was fairly substantial. On a 12 in. × 12 in. pile it would be 180 tons at the toe, and if there were added a modicum of friction such piles would carry something like 200-250 tons. If the authors were getting 180 tons on the toes of the piles and were loading them only to 41·5 tons, he was wondering whether piles were necessary. He wondered if it would not be possible on the sand at this site, remembering that it had been de-watered and therefore improved immeasurably, to support the structure without piles. Alternatively, it might have been possible to use vibro flotation, which might very well have enabled them to avoid using piles.

As to the driving of the piles, he had tried to calculate what the stresses might have been, because he had noticed that the authors did not use pile shoes; he would think the stress at the toe was well over 2,000 lb/sq. in., probably 2,500 lb/sq. in., and that was rather high for the toe of a pile which one could not see.

It had been stated by the authors that when they had dug deep pits near the piles they had stopped driving and had got down into the pits. That suggested to Mr Bullen that the sand was really firm; so that again he asked why they had piled. He also would like to know whether any re-driving tests were carried out on the piles because in some sands a high initial driving resistance gave place to a low subsequent resistance, often called a 'take-down.'

THE PRESIDENT said he was intrigued by the dredging operations on the site, and he asked if the authors would say what had happened to the lagoon which they had left one and a half miles away from the building. Did it now form part of the River Dee?

The discussion, he added, had been very interesting and he was sure the authors would welcome the comments that were made.

Mr. W. T. BROOKS, replying to the question whether the floors were suspended or merely placed on sand, stated that the floors were just laid on the sand which had been compacted with a heavy vibrating roller. The pile caps formed hard spots and were therefore completely surrounded with expansion joints. The layout of the floor slabs was such that where the columns were 45 ft. apart, three 15 ft. wide slabs were laid, one of the expansion joints across the shop being at the column centres. The length of the main slabs varied according to the shape of the Plant foundations in the bays, being in general between 17 ft. and 20 ft. The area between columns was covered with slabs approximately 13 ft. long and exactly the same width as the transverse length of the pile caps. In designing the pile caps care was taken to keep them from protruding into the bays as far as the nearest position of the E.O.T. crane hook. By doing this the pile caps and the floor slabs between caps were generally unloaded as far as heavy storage loads were concerned, but they could be loaded from moving traffic between the bays.

A large amount of research had been done on the soils in the Shotton area, the grading curves for the site sand showing that 90 per cent of the grains were between 0·05 and 0·3 mm in diameter. These grain sizes were considered to be very fine and it was doubted if the sand had the inherently high internal stability characteristic of granular materials. There had been a possibility that it might tend to act as a cohesive soil. However a soil survey proved this supposition to be incorrect as it was found that the site sand was stable. One of the tests was carried out on a 5 ft. square test bed 6 in. below ground level; this slab when loaded to 1·8 tons per sq. ft. showed a settlement of $\frac{1}{4}$ in. It was expected that the floor slabs in the E.R.W. Mill would have to carry storage loads of up to 1 ton per sq. ft. but on looking into this further the actual storage developed over the whole floor slab would be about 0·75 ton per sq. ft. It was therefore expected that the settlement on floor slabs would be from $\frac{1}{4}$ in. to $\frac{5}{16}$ in. In the adjacent steelworks many floor slabs had carried loads of 4 tons/sq. ft. for many years, producing a settlement of between 4 in. and 5 in. One of the reasons for keeping the floor slabs between the columns the same width as the pile caps was that these slabs were unlikely to settle as the areas immediately adjacent to the main columns were normally used as walkways; eliminating the possibility of trip hazards due to the pile caps standing proud of slabs. The 8 in. thick floor slabs were reinforced top and bottom and the concrete base slab and granolithic surface were laid in one operation. Trentham gravel was used for the granolithic surfacing, as by test this had been proved to be approximately twice as resistant to abrasion as local materials. The base concrete was internally vibrated and the granolithic topping was surface vibrated and trowelled up to a very fine finish.

'Lithurin' is a proprietary floor hardener. The compound should be applied *more* than 14 days after the floor has been laid.

On two occasions in the past in other Works Mr. Brooks had had to supervise the laying of granolithic surfacing on to an already hardened concrete floor slab; on each occasion a different patent compound was used in an endeavour to fix the topping firmly to the base slab. On both occasions the topping cracked up in a very short time. In his opinion granolithic topping could not be satisfactorily laid unless monolithic with the base slab.

Mr. Boyse had mentioned that there had been discussions between the steelwork designers and those responsible for the foundations. Large moments had been exerted at the base of the longitudinal surge columns shown in Fig. 5 of the paper. The columns were entirely cantilevered about the pile cap and the lateral moments had to be resisted by the piles and the weight of the pile cap. It had been suggested that portal or other bracing might be used to avoid these high moments but as the space between columns had to be kept clear for the easy passage of tubes from bay to bay this could not be done. In every other respect there was no reason to suppose that the moments induced in the foundations were greater than in any other similar structure built with other materials.

Mr Bullen had referred to the test pile mentioned in the Paper. As the Paper had had to be condensed,

it was not possible to give all the test data and only the general results were included. The helmet packing used in driving the test pile comprised a 3 in. thickness of laminated plastic with a 1 in. thick mild steel plate on top. Soft wood packing, 2 in. thick, was used for immediate contact between the helmet and pile head, but this packing was thoroughly compressed with a few blows of the hammer. The test pile which was used was 25 ft. long and 14 in. square and was driven from a level of + 28 *O.D.* through 14 ft. of dredged sand filling over the natural marsh level. It was driven by 10 blows of a 3 ton drop hammer to a ¼ in. set, which as Mr. Bullen pointed out was possibly overstressing the toe of the pile. He agreed that it was dangerous to drive a pile to this set under normal working conditions, but this test pile was especially driven to virtual refusal on the clients' instructions. Usually a set of 0·5 to 0·7 in. was used. It was usually easy to discover if a pile had broken below ground level as it very quickly sheared away from the vertical and when this occurred one or two of the main reinforcing bars appeared to break bond and pushed through the head of the pile a small distance into the soft wood packing. It was apparent on inspecting such broken piles that they tended to shear off some 25 to 30 deg. from the vertical. Often a break in such a pile occurred just above ground level when the pile was rather more than half driven, the break usually being caused by excessive movement of the pile frame subjecting the pile to bending and impact stresses at ground level. This was especially so when driving by drop hammer, but the innovation of the Delmag Diesel Hammers and Frames had eliminated all movement due to vibration. Assuming the test pile to be a short column unrestrained at the sides with a concentric load of 130 tons and an *E* value for the concrete of 2,000,000 lb. per sq. in., then the pile would have compressed 0·198 in. The test pile actually compressed 0·12 in., the difference in compressive strain being 0·078 in. Presumably this difference was due to the side shear on the pile in the ground, the resistance from friction amounting to 51 tons. This may not be an accurate assumption, but presumably the rest of the load is resisted at or near the toe of the pile. Mr. Bullen had suggested that this would overstress the toe of the pile, but Mr. Brooks did not feel that this was necessarily so, as the loading would be almost perfectly tri-axial. In the Mill building under consideration it would not have been possible in the design of the pile caps to use 14 in. square piles with a working load of 124 tons, nor 12 in. square piles loaded say, with 90 tons, giving a deflexion of 0·12 in., as when the piles under the main crane columns were fully stressed on the crane rail side they were only stressed to 2 tons maximum load on the other side. As a result the cap would tend to rotate 0·12 in. on one side only and as the inner and outer piles were 10 ft. apart the crane rail would tend to rotate three times this deflexion, i.e. about 0·36 in. The distance between crane rails could therefore be reduced by approximately ¾ in. and large maintenance expenses could have been incurred due to excessive wear of crane rails and crane wheels.

It was Mr. Brooks's practice to use 14 in. square piles to take dead loads of 65 tons, this loading being increased to 75 tons with induced load from moments, the main purpose being to avoid too much movement at crane rail level, especially as the crane rails were 60 ft. above floor level in some of the steelworks with which he had been associated.

12 in. square piles were used in the new Mill at Shotton because the carrying capacity of 14 in. square piles was too large to be economical in this instance.

Mr. Bullen had asked why piles were used instead of deep slab foundations or ground compaction by the vibro flotation method. The site had been surveyed and loading tests had been made. When a load of 4·5 tons per sq. ft. was placed 6 ft. below marsh level, on top of which there was 14 ft. of dredged filling (a total wellpointed depth of 20 ft.), the settlement varied between 0·18 and 0·73 in. It was also found that a load of 6·4 tons per sq. ft. placed on a 5 ft. square test bed 6 in. below the surface produced a settlement of 6 in. This merely confirmed the view that depth was important as far as the load bearing capacity of the ground was concerned. Although much of the peat had been cleared by drag line, it was not economical to clear the whole area. In constructing the Mill foundations the invert of one pit was 22 ft. below the surface and a peat layer from 11 in. to 15 in. thick was encountered 3 ft. below the thin black line which indicated the original top soil level of the marsh before the site was filled. Piling was used because it was cheaper than a deep foundation, which, because of the possibility of encountering peat at or below top soil level, would have had to be taken down 12 or 13 ft. below the surface. Had a deep foundation been used well points would have been needed and there would have been much more excavation, together with a longer concrete plinth to bring the foundations up to the underside of the column; this would have meant more dead weight on the base.

With the exception of the large columns shown in Fig. 5 of the Paper, five 12 in. square piles were used under the more heavily loaded crane columns in the building. The average length was 22 ft. and the cost per pile slightly less than £19. The cost for foundations for all the buildings amounted to 3·49 shillings per sq. ft., this figure including all piles, caps and suspended crane beams to carry dwarf and other brick walls throughout the whole building. This cost allowed for the 14 in. of concrete in the top of the caps, for the precast concrete bolt tubes and for grouting up, but excluded the cost of the holding down bolts and the anchor frames.

All the piles were driven to a set of 0·7 in. from 10 blows of a 30 cwt. hammer. The 'revised' Hiley formula used by Holst & Co., was as follows:—

$$R_2 = \frac{W^2 H n}{\left(W + P\right)\left(s + \frac{c}{2}\right)} + (W + P)$$

Where,

R_2 = ultimate resistance of the ground in tons.

W = weight of hammer in tons.

P = weight of pile and helmet in tons.

s = set per blow in ins.

H = height of fall in hammer in ins.

c = temporary elastic compression in ins. of the cap and pile and ground into which it penetrates.

n = a factor based on the efficiency of the blow, corresponding to $0{\cdot}8\left(1 + \frac{P}{W}\right)\eta$ in Hiley's original paper.

The following table may be derived for use in design :

$\frac{P}{W}$	n		Pile length in ft.	$\frac{c}{2}$ in ins.
	$e = 0{\cdot}4$	$e = 0{\cdot}25$		
$\frac{1}{2}$	0·87	0·83	10	0·11
1	0·92	0·85	20	0·13
$1\frac{1}{2}$	1·00	0·88	30	0·15
2	1·06	0·90	40	0·17
$2\frac{1}{2}$	1·11	0·95	50	0·19
3	1·15	0·96	60	0·21

In this table,

$e = 0{\cdot}4$ for hard packing.

$= 0{\cdot}25$ for less hard packing.

The compression values used in deriving c are :

0·04 in. per 10 ft. length of pile.

0·085 in. in the helmet and packing.

0·085 in. in the ground.

It was considered that an instantaneous value of E_c of $4{\cdot}5 \times 10^6$ was more realistic than the 2×10^6 used by Hiley.

To arrive at a working load, R_2 must be divided by a factor of safety, which may be 2 to $2\frac{1}{2}$ when piles are designed for a maximum compression of 680 lb/per sq. in. or from $2\frac{1}{2}$ to 3 when the maximum compression is 600 lb/per sq. in. or less. The factor of safety can be reduced in a long pile owing to the transmission of the blow as a wave, rather than instantaneously.

In the past 22 years, 35,000 piles had been driven in the area and no settlement had yet been recorded in a piled foundation, not even where floors were loaded to 4 tons per sq. ft. Although there had been no re-driving at the new Tube Plant, there had been plenty of tests elsewhere in the area. Sometimes, piles were excavated to half their original depth and re-driven with the frame bridged on a small gantry. It was necessary to apply 50 to 100 light blows of the hammer to get the pile started and from the sets measured it was apparent that the lower half of the pile was resisting at least 85 per cent of the loading. However, no load tests had been made with the pile in this condition. When attempting to drive from surface level, it was usually found that very little movement could be obtained in a pile originally driven to a set of 0·7 in., as already mentioned.

One advantage of using piles at the Tube Plant was that the vibration had a consolidating effect on the general filling. Over the main gutters a maximum depression of $2\frac{1}{4}$ in. was recorded, partially due, no doubt, to the existence of layers of peat.

The factor of safety against settlement in the piled foundations exceeded 2. In any slab foundation there might have been differential settlement.

A tri-axial compression test under laboratory conditions had been made on the site sand. When a tri-axial load of 1 ton per sq. ft. was applied, the void content was 40 per cent. When the load was increased to 5 tons per sq. ft., the void content was reduced to 38 per cent. In driving 12 in. square piles at 3 ft. centres, the void content was reduced by 11 per cent.

When excavating beside piles it had been found that the sand was extremely hard. It was therefore felt that the subsequent 'take-down' mentioned by Mr. Bullen would not take place.

Vibro flotation might have been effective, but not necessarily economical. When using this method, however, the sand is not consolidated evenly over the whole area vibrated. Soil tests have shown that the bearing capacity of the sand is sensitive to changes in void content. Furthermore, the buildings were half erected before the designs were completed for the actual mill foundations, most of which proved to be very deep, eleven separate well-point operations being required in the bays after the building foundations had been completed. This could have had a detrimental effect on sand treated by vibro flotation, as the reduction in void ratio would not have been as much as that caused by piling. The existence of layers of peat could also reduce the consolidation effect of vibration.

To sum up, it is felt that all large buildings carrying E.O.T. cranes should have positive foundations to prevent possible expense at a later date.

As all the rainwater was taken to external down-pipes there was no necessity to provide storm water drains beneath the buildings. Two main outfalls discharging into the west channel drained the west side of the buildings and the main storage areas. Two smaller outfalls laid from the east channel drained the east side of the buildings and the carriageways in that area.

In response to the President's question about the lagoon left by dredging operations, the dredging was not carried out in the bed of the River Dee, where the river bed would have been completely backfilled within a few weeks, but in the large area of saltings flanking the river. The dredger which could only be brought over the saltings on two of the highest tides of the year, had excavated a lagoon about 400 ft. in diameter and 35 ft. deep at the completion of the operations. Within two or three years the lagoon will be about 8 ft. deep and the surrounding saltings will be lowered by a few inches.

Mr. G. B. GODFREY, in reply to Mr. Bondy, stated that although E.R.W. tubes were among the five types permitted by Clause 3a of B.S.449 : 1959, they had until now been produced in sizes not exceeding $4\frac{1}{2}$ in. o.d. and were normally employed for rather more *de luxe* purposes than structures. The bulk of the tubes used for structural purposes were continuous weld (Fretz-Moon) tubes, also not exceeding $4\frac{1}{2}$ in. o.d., which were cheaper than E.R.W. tubes, and seamless tubes, not exceeding 18 in. o.d., which were also more expensive than continuous weld tubes.

Being made from strip, the new large diameter E.R.W. tubes would be limited in wall thickness, but provided they compared favourably with seamless tubes from the technical and economic points of view, it was reasonable to assume that they also could be used for structural purposes.

There were no physical reasons why E.R.W. tubes should not be made into R.H.S., but again their use would depend on their selling price.

Mr. Godfrey welcomed Mr. Bondy's confidence in the use of tubes in surge girders. Compound rolled sections or plate girders were used for the short crane girders shown in Figs. 7, 10, 12 and 14, but lattice girders were used for the 60 ft. and 90 ft. spans, R.S.J's being used for the top chord carrying the crane rails and tubes for the bottom chord and web members, as shown in Fig. 13 of the paper. For long spans there was no reason why such a lattice girder should not compete with other forms of construction.

Dr. Terrington had expressed some concern about possible fatigue effects in the crane and surge girders. The tube interests in Great Britain had had considerable experience of large tubular crane jibs and also of tubular surge girders but this was the first occasion when they had used tubes for the gantry girders, although the practice was common abroad. As a result, the design loads had been increased by 50 per cent to allow for dynamic effects, instead of the usual 25 per cent

customary for such conditions. The designers were quite confident that the tubes would behave efficiently in practice.

Dr. Terrington had mentioned the roof of the Admiralty experiment tank at Haslar which had been described by Hunter, Little and Hacon in the *Proceedings of the Institution of Civil Engineers* for January 1959. There, very considerable savings in steel had been made by employing tubes for the main girders, which were 218 ft. in span. As the ratio of dead load to live load normally increased rapidly with the span, cumulative savings could be made by employing light media, such as tubes. Savings were also made at Shotton, but as the spans were much less spectacular, they were more modest than at Haslar.

Mr. Godfrey observed that he had been a witness on a number of occasions at the presentation of papers to the Institution when members had made fruitless attempts to obtain information on costs. For that reason, the authors had tried to give appropriate information in their paper. It had occurred to them, however, since the paper was published, that there was some ambiguity in that there was no roof over the coil stock bay. If covered structures only were considered, then the cost per sq. ft. of the framework was 8 per cent more than that stated in the paper.

Mr. Bullen had asked about final costs. There had been a number of variations in the shape and layout of the buildings, but the final unit costs would be very little more than those quoted in the paper.

Being in the fortunate position that they themselves had not designed the buildings, the authors were able to survey them with critical eyes. Observations had been made on the foreshortening of the bottom booms of the crane gantry lattice girders. Nevertheless, those whose duty it was to use the works had already commented on the smooth running of the overhead cranes and the lack of noise and vibration.

Mr. Bedford had suggested that continuous construction might have been used with advantage for the crane girders. Mr. Godfrey felt that with experience of this type of construction the designers would become more ambitious. Nonetheless, in general he preferred cantilever and suspended span construction to statically indeterminate construction.

The authors in general expressed pleasure that Mr. Boyse had taken part in the discussion, the more so because he had answered the questions about the north-light roof.

There were cases in the plant where it would have been slightly cheaper to have used rolled sections for columns, but in a building mainly tubular in construction it was reasonable from the practical and aesthetic point-of-view to use tubes throughout.

With regard to the possible use of rectangular hollow sections, it could be stated that R.H.S. were never produced to replace tubes, but to supplement them. The main advantages of R.H.S. were that they were more efficient in flexure and that they provided flat surfaces, a fact which led to their being used by fabricators not equipped to 'saddle' the ends of tubes for jointing. Mr. Godfrey therefore felt that there would have been little to gain in the use of R.H.S. in the roof of the Shotton plant, while the R.H.S. at present produced were not sufficiently large for use as columns.

The most important phrase in Mr. Tietz's question was "some years ago." There were *now* many firms in Great Britain who would be prepared to quote for a steel structure embracing conventional rolled sections, tubes and R.H.S.

Mr. BURNETT-STUART, replying to Mr. Bondy, stated that the site welds were examined visually. The site welding was not extensive, being restricted to the north light girders, which were delivered in two halves, and to the 90 ft. girders. The three men employed on site welding were Lloyds-tested and as they had all been employed on the Haslar scheme, with which Mr. Bondy had been associated, Mr. Bondy would probably agree that they were highly-skilled.

Book Reviews

Influence Lines for Continuous Beams, by Kerry S. Havner and Jan J. Tuma. (Oklahoma State University Engineering Experiment Station Publication No. 106, September, 1959). 50 pp. $ 1.50

The application of the algebraic carry-over moment method to the analysis of multi-span continuous beams of constant or variable cross-section is presented in this publication. The general three moment equation is the mathematical model used and the infinite, convergent, geometric series the means of solution.

The derivation of the three moment equation for beams of variable section, the theory of algebraic carry-over procedure, and the derivation of moment influence coefficients in algebraic form are shown in Chapters 1 and 2. Numerical examples illustrating the application of these influence coefficients are included in Chapter 3. The Appendix contains two sets of moment influence tables which are especially suitable for the analysis of continuous beam bridges.

An Experimental Study of a Quarter-Scale Reinforced Concrete Flat Slab Floor, by D. S. Hatcher, M. A. Sozen and C. P. Siess (Urbana, Illinois: University of Illinois Civil Engineering Studies Structural Research Series No. 200, 1960). 11 in. × 8½ in., 134 pp., 121 figs.

This report describes one phase of an investigation of the behaviour and strength of multiple-panel reinforced concrete floor slabs. The investigation is concerned with two general types of floor slabs commonly used in building construction, i.e. slabs supported on beams spanning between columns and slabs supported on columns only, and static tests on quarter-scale models of typical structures are being carried out. The present report describes tests on a quarter-scale model of a flat slab and three of the thirty-eight tests carried out are described, these are the design load test, the overload test and the test to failure, with all panels loaded in each test.

The Structural Engineer in the Coal Industry *

Discussion on the Paper by C. A. C. Davies, M.I.C.E.

Mr. K. H. BEST (Member), said Mr. Davies had summarised the factors affecting the design of winding towers and had pointed out that these structures presented unusual and interesting problems, for which there were no standard solutions; and there was very little published information.

Before completing a design, the structural engineer must therefore combine the results of theoretical analysis with a good deal of engineering judgment.

Mr. Best said he believed the National Coal Board issued draft regulations in respect of non-structural aspects of winding towers, and he wondered whether Mr. Davies and his colleagues were considering the preparation of a draft code of practice for structural design.

A fair number of tower mounted winders were now in commission, so that much practical and theoretical data should be available.

Inviting the author's comments on one or two design points where opinions varied, he said that with regard to vibrations it was impractical to attempt to calculate the natural frequency of the whole tower structure above its foundations, and he suggested it was only necessary to check those members directly supporting rotating parts. One criterion was to proportion such members so that their natural frequencies were greater than three times those of the rotating parts, and fractional. With concrete towers there was usually no difficulty, but with steel towers, although the main bearing girders were generally stiff enough, it was usually essential to check motor and gearbox supports. For example, in the Kellingley towers (Fig. 5) the plate girder system supporting the motors was substantially stiffened to meet the frequency requirement. Perhaps it was as well that they were because, during commissioning, the twin motors were initially installed out of phase, setting up severe out-of-balance vibrations, for which, incidentally, the structure was blamed, until wiser counsel prevailed. There were usually teething troubles during commissioning, and a tower designer should not be unreasonably alarmed when, after the plant was first started up, he was told that his structure was wobbling, because the cause would be an unbalanced drum or machine in most cases.

Other interesting points were concerned with the top section of a guide tower, housing the retarders and catchgear. At the top of page 373 Mr. Davies suggested that although this was usually regarded as shaft equipment, it was very much within the sphere of the structural engineer. Mr. Best went further than that and said that it was an essential part of the design of the tower, to which it should be connected, rather than comprising an independent structure.

Reference to Section B-B in Fig. 3 would illustrate that point, and he thought the possibility of the conveyance jamming in the retarders should be considered, especially where these were timber taper guides. If this happened below bumping beam level, reactions between drum and conveyance would still be balanced internally when the top of the guide frame was connected to the tower structure, but if it were separated—for example, supported on the shaft collar—both main bearing girders and guide structure might separately suffer unbalanced rope load.

Although rope breakage was a remote possibility, overwinding did sometimes occur and the safety catches could operate more frequently—for example, if a rope came off the deflector pulley during overwind. He had seen that happen. A fully loaded conveyance, dropping back two or three inches on to catches, gave severe dynamic loading, and the engineer had to judge a compromise between sufficient flexibility in supporting steelwork—which would reduce the shock load by absorbing energy—and required strength.

Practice varied considerably, but in at least one area the catches were suspended from the guide frame by spring loaded shock absorbers. That system enabled the loads to be determined far more accurately and gave a more economic structure, and Mr. Best suggested that it might be adopted more frequently.

Finally, he congratulated Mr. Davies on his most interesting paper, and especially on his excellent illustrations.

Mr. DAVIES agreed with all that Mr. Best had said. It was quite true that there were no standard solutions to the problems of winding engines; in fact the development of modern winding engines had taken place very rapidly during the post-war years and he did not think it had been possible to achieve a standard solution. But now there was a great deal of accumulated experience and consideration was being given to putting it, not into a code of practice, because that was apt to give too much finality, but at least into a manual which would give some guidance in the design of the supports of winding engines and gearboxes.

Vibrations presented a real problem and the Board had consulted Professor Grootenhuis of the Imperial College of Science and Technology about it. His recommendations, some of which Mr. Best had stated, were now used in design.

This problem also applied to Coal Preparation Plant structures. In the latter case it could be more difficult to deal with because it was possible for additional vibrations to be induced in order to improve the efficiency of the screens. This was difficult to cope with in design.

On the question of balanced reactions the majority opinion in the Board was that the guide frame and bumping beams should be attached to the underside of the beam system supporting the deflector pulleys and winding engines so that in the event of an overwind resulting in impact on the bumping beams the reactions were balanced internally.

Some engineers favoured an independent guide frame structure but the author agreed with Mr. Best that this might induce unbalanced rope loads under overwind conditions.

It had been said, quite rightly, by Mr. Best that this was very much a structural problem, but nevertheless it is one which is closely allied to mechanical and electrical engineering problems and there were still many mechanical engineers in the Board who regarded the guide tower and indeed a headframe as a mechanical structure. The structural engineers were bound up not only with the mechanical engineers, but with the mines inspectors, who were very concerned with the safety conditions at the shaft top. New developments and devices had to be very carefully examined from this point of view and it might well take a long time to reach a decision. He considered that it was right that this should be so.

The question of resilient catchgear had been raised by Mr. Best and he believed that in some cases spring

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loaded catches had been used by the Board. He agreed entirely with Mr. Best that it was a very good thing to introduce this resilience, for it did help in structural design. However, this was again a matter which lay outside the jurisdiction of the structural engineer who would only put forward the design advantages of such an arrangement.

Mr. W. G. TAYLOR expressed his appreciation of a very good paper indeed and a good film.

Digressing a little from the points raised in the paper, and dealing with the point as to how the mining engineers could co-operate with the structural engineers who had to work on the surface, he said there was the basic problem of disturbance on the surface, and in some cases headframes had to be designed so that they could be corrected back to the vertical.

We knew that in the Nottingham area some years ago efforts were made on other than industrial buildings to provide a very flexible pin-jointed frame which over the years could be corrected to counteract any change due to ground movement.

We also knew that for many years the reinforced concrete raft had been used to minimise such changes and he asked Mr. Davies whether any new system were envisaged by the Coal Board.

One problem was to restore the correct position of a building where there was not a resident engineer. Another was the determination of the possible degree of movement due to subsidence.

With modern mining methods, it was said in some quarters that the effect on the level of the surface was more than it used to be, but to his simpler mind it seemed that the change in level came about more rapidly in these days than formerly, but that the overall change in level was no greater.

Mr. DAVIES said that generally speaking the structural engineers in the Board were not faced with subsidence problems as most of their work was built on the shaft pillar. It was a problem which lay outside his own sphere and he would not like to become involved in a discussion upon it. Naturally he and his colleagues were interested in the structural aspects of it and were aware of the value of flexibility.

Mr. TAYLOR questioned the degree of subsidence. That depended upon the depth and the number and thickness of the seams being worked and he would certainly not say that it was any greater now than in the past. If anything it would perhaps prove to be less as thinner seams were now being worked. Mechanisation produced a more rapid movement of the working face and the rate of subsidence was thereby increased.

Mr. M. NACHSHEN, (Member), commented that the very interesting film which the meeting had seen did not show how the concrete of each ring was cast up against the underside of the ring previously cast. The method used would be of interest for other applications.

Mr. DAVIES replied that when he had first seen the film he had thought it a little deficient in that it did not show the actual concreting.

In this rapid method of sinking, the walling length was not more than 15 ft. when the shuttering was brought up to the underside of the concrete which had been placed, and it was not quite clear how that was done. But normally, at the junction of the old concrete with the new, the shuttering was brought above the existing concrete, so that a head was introduced which forced the concrete up underneath the existing concrete. It did not form a very water-tight joint. The Parkside sinking was not through heavily water-bearing strata.

Mr. J. B. WEST wondered whether high alumina cement was used for the walling, because of the sulphates. He recalled having read a paper some time ago about a shaft in which there was a great deal of sulphate trouble and it had practically to be re-lined. Perhaps Mr. Davies could say something about that trouble.

Mr. DAVIES said that high alumina cement had been used for the walling of shafts, but it would not be customary to use it. A very large volume of concrete was involved, and it was normal to use ordinary Portland cement. The trouble to which Mr. West had referred did certainly take place, and high alumina cement was used because of its sulphate resistance.

There was a certain amount of garbled reporting about it in the press, which had left one with the impression that if sulphate resisting cement had been used there would not have been trouble. This was not true. However, he was not the engineer responsible for the work and it would be wrong for him to go further into it on this occasion.

Mr. B. L. CLARK (Member), commented that we could always learn from the work of others; the Paper by Mr. Davies was certainly in this category and was most interesting.

He had gathered, maybe wrongly, that the blasting was done when the concrete was only eight hours old. If that were so, he wondered if any troubles were experienced.

Mr. DAVIES said it was quite true that they did blast when the concrete was only eight hours old; so far as he was aware, no trouble had been experienced. Taking refuge in the fact that he was not closely associated with this work, he said he was not able to elaborate on it.

In the majority of shafts that were sunk in this country the walling lengths were of the order of 80ft. and consequently the concrete in the vicinity of the blast was very much older.

THE CHAIRMAN asked, was not the steel shuttering in position when the blasting took place, and did that not protect the green concrete from the effects of the blast?

Mr. DAVIES believed that to be perfectly true. One would have thought, he said, that the effect of the blast could communicate itself through the shuttering and into the concrete; but at Parkside there had been no apparent disturbance.

Mr. CLARK asked what was the weight of charge, and what "jelly" (explosive) was used.

Mr. DAVIES said he did not know the answers off hand, but would try to give them in writing.

A SPEAKER asked if any special precautions to deal with water were applied in conjunction with the method of sinking shown in the film.

Mr. DAVIES said that the Board normally used the freezing method when sinking shafts through water-bearing strata. At Parkside some water was encountered as could be seen from the film, but the strata was not water-bearing in the sense, for instance, that the Bunter sandstones are and the water could be controlled by pumping.

Where shafts are sunk through water-bearing sandstone which might go down as much as 1,000 ft., two processes were normally used; they were freezing and cementation. The Board had in most of its later sinkings adopted the former. By this method a number of circumferential holes were drilled through the water-bearing strata and into impervious formations below. The holes were lined with steel tubes and chilled brine circulated in them until an ice wall was built up.

THE CHAIRMAN, at the conclusion of the meeting, thanked Mr. Davies for the admirable way in which he had replied to questions, and also all those who, by contributing, had made the discussion so interesting and informative.

Rubber Tyred Overhead Cranes and their Influence on the Design of Industrial Buildings *

Discussion on the Paper by E.M. Lewis

THE PRESIDENT introduced Mr. Lewis who then presented his paper.

Among his slides there were illustrations, from a book published in 1908, of cranes which were dated about 1890.

He also showed a film to clarify some of the technical points in the paper, including the operation of the rubber tyred crane with side rollers to keep the wheels on the track.

Discussion

Dr. H. R. MILLS said he had read the paper with great interest; but while agreeing that there were sufficient advantages to be derived from the use of rubber tyres on some types of crane to justify their wider use, he could not wholly share Mr. Lewis's enthusiasm for this development. If he raised certain points against the use of rubber tyres it did not mean that he did not believe in them; but he felt it should be made clear which of the advantages claimed for this development were essentially due to the use of rubber tyres and which were due to other improvements which could be introduced whether rubber tyres or steel wheels were used.

The crane Mr. Lewis had illustrated in the film had not only rubber tyres, but was fitted with side rollers to keep it on its correct path and had independently driven wheels on the two sides of the crane. Side rollers had to be used, of course, with rubber tyred wheels, and they presented far less resistance to motion than the rubbing of steel flanges against the side of a rail, but there was no reason why side rollers should not be used with flangeless steel wheels. Neither was there any reason why a steel wheeled crane fitted with side rollers should not have independently driven wheels without any cross shaft connecting them. Mr. Worthington, indeed, in a paper to this Institution in 1957,[3] had recommended the use of cross shafts, but had stated that, where they were not used, good results could be obtained with side rollers. Furthermore, Dr. Mills had seen a steelworks crane with independently driven wheels and guide rollers in Germany even earlier than that (in 1955 or thereabouts) and he understood that it had operated successfully; that crane also ran on a rail mounted on a rubber strip to reduce impact loads. He suggested that if we wanted to know the real advantages of rubber tyres for cranes we should use a crane such as that German one as our standard of comparison. If we compared that kind of crane with the rubber tyred crane described by Mr. Lewis we were still left with some very real advantages. One was the reduction of impact loads, and another was the ability to run directly on the girder without a rail, so avoiding all the problems of rail fixtures and joints. But the ability to run satisfactorily on an imperfectly aligned girder was, to his way of thinking, due to the use of guide rollers rather than rubber tyres.

There were also certain disadvantages of rubber tyred cranes which were not mentioned by Mr. Lewis. Steel wheels of 30 in. diameter frequently carried loads of about 8 tons per inch of rail head width; that was 24 tons per wheel on a 3 in. rail, a quite common size in steelworks. What load could be carried by a rubber tyred wheel of the same outer diameter? Was it not a fact that for all but very light cranes the number and size of wheels would be unacceptable, necessitating a very long end carriage and increased height clearance in the building?

In his view the real advantage which rubber tyres gave to mechanical handling plant in general was mobility. In works, as on the public roads, it was not practicable to provide a surface comparable with that of a pair of rails. If, therefore, we no longer wanted to be bound to follow rail tracks, we must overcome the irregularities of the surface on which the machines run by using rubber tyres. It was this "go anywhere" mobility with reasonable freedom from shocks which had led to the growth of road traffic at the expense of rail and he agreed with Mr. Lewis that the same trend was discernible, and was to be encouraged, in the handling of materials in works. Mr. Lewis might even be right in thinking that in some areas of the works—he had instanced a slab yard as one—a rubber tyred Goliath crane was an attractive alternative to an overhead crane; after all, this was only a larger version of the straddle carrier, which was used for similar purposes on somewhat lighter material than steel slabs. One must not forget, however, the main advantage of overhead cranes running on a girder within the building, namely, the ability to transport material between any two points, regardless of what else was in the building and without the necessity to keep any floor space clear for the passage of wheels. The overhead crane was a convenient means of transport within a single bay, but it was the transfer of material from one bay to another which was difficult. The rubber tyred vehicle could perhaps solve this difficulty in a more satisfactory way than the conventional steelworks method of using a railway for the purpose, and this might well lead to a different layout, with a sequence of processes taking place in adjacent bays instead of in separate buildings joined by railway lines. Some of the more recent steelworks layouts did in fact show a trend in this direction.

Mr. LEWIS replied that the advantage of using guide rollers for steering steel-wheeled cranes was that there was not a flange which might bind but such cranes did demand a similar degree of track accuracy to that which was necessary where flanged wheels were used—nothing was gained in the steerability of the crane. As was stated in the paper when the angle of attack between a steel wheel and a crane rail was of the order of two in a thousand the wheel might begin to skid; but with rubber tyres we could go to five or ten times that value without skidding. In one of the first applications of the guide roller principle in this country it had taken two years to get the cranes to run

* *Read before the Institution of Structural Engineers at* 11, *Upper Belgrave Street, London, S.W.*1., *on the* 23*rd March*, 1961. *Lt.-Col. G. W. Kirkland, M.B.E.(Mil.) M.I.Struct.E., M.I.C.E. (President) in the Chair. Published in "The Structural Engineer" Vol. XXXIX, No.* 3, *pp.* 100-106.

smoothly, and he believed that on the first occasion when the cranes were run the guide rollers literally fell off, due to the large steering forces.

From the steelworks point of view an improved layout was an important matter, because it might make all the difference between using one crane or two ; we could have one crane doing the job instead of two, eliminating what was virtually a standby crane, and that would save a lot of money.

On the question of the load which we could put onto a rubber-tyred wheel, a steel wheel could carry 20 to 30 tons on a 3 in. rail and this was one of the awkward limitations of the steel wheel. When using rubber-tyred wheels instead of putting all the load onto a width of 3 in. we could spread it over a 30 in. width if we wished. We could put the same load per wheel on rubber tyres as with steel wheels because not only could we increase the bearing width, but also by reason of the elasticity of the rubber the tread contact length was also increased. However, he thought it fair to say that at this stage end carriages with rubber-tyred wheels would be somewhat larger than those with steel wheels.

Reference was made in the paper to the break even length of buildings. In a long building it was much cheaper to have a rubber-tyred crane than one with steel wheels ; in a short building the extra cost of a rubber-tyred crane was not sufficiently offset by what was saved on the girder and supporting structure.

With regard to the rubber-tyred crane which could be steered from shop to shop, he was very glad Dr. Mills felt that there was advantage in that respect and he had added a touch of respectability by saying it was just a big straddle carrier. Mr. Lewis commented : " Let us look on it as a big straddle car, but let us have it."

Dr. J. S. TERRINGTON (Associate-Member), discussed the effects of impact and the local wheel effects, and began by asking Mr. Lewis what value of the impact factor he would take on rubber tyred wheels.

Referring to what, he said, might be termed past history he recalled that the impact factor for gantry girders was specified in the present British Standard 449 as twenty five per cent of the static wheel load and that, in point of fact, it originated in a recommendation made in a report on Loads and Stresses in Gantry Girders published by the Institution in 1927. The value of the factor was nominal.

Coming, however, to developments over more recent years, as a result of numerous structural tests carried out, he said, under operational conditions in iron and steelworks by the British Iron and Steel Research Association, the nature of the loading was now quite clear. The impact effect was not, as we might imagine and as implied by B.S. 449, a complete total rise of twenty five per cent on the static value, but it was in fact, a slight ripple of five or ten per cent above and below the static value. This has been shown on the transient recorded traces of strains induced in the girder, as the wheels of the end-carriage of the crane moved over the girder. He illustrated this by a sketch showing typical outlines of traces of strains recorded from electrical resistance strain gauges when they were attached to a span of a gantry girder, at mid-span in the top flange and near the support of the web. These recorded traces, he said, conformed in fact to the influence lines of the stress in the girder at the point of attachment of the strain gauge, and conformed to the static load of the crane wheels as they traversed the span of the girder. Each trace was typical, namely, in the one case of the variation of the boom stress in the flange, and in the other case of the shear stress in the web.

In addition to the impact ' ripple,' the flange and web immediately under the wheel were severely depressed, causing intense local effects. This was shown by marked peaks and troughs in the recorded trace.

Contrary to the impression generally held, the impact effects in the recordings due to rail joints were of little consequence and may have earlier been confused with the local effects. Dr. Lightfoot's work seemed to show that these effects on the crane bridge were not high and the effects on the gantry girder would be correspondingly less serious.

Mr. LEWIS, speaking of the local effect, thought we could say that the best approximation we could make when using rubber tyres was zero ! He did not know whether the nominal impact factor had any meaning for rubber tyres. He had been honest and had been very careful to say in the paper that we did not understand this ; and we didn't. We knew that when a crane suffered an impact a shock wave was sent through the crane and it could be of the order of 100 g. But the amplitude of this oscillation depended upon the mass spring system ; at this juncture he thought we could say that, when using rubber tyres, the mass spring system was different from but no worse than with steel wheels. With the advent of the broad flanged beams the form of construction was very simple and cheap, but it brought in transverse bending stresses which were greater than they were with steel. He was concerned with some fatigue tests being conducted by the British Welding Research Association in this connexion and which would no doubt be reported in a future paper.

Mr. D. MANOLOPOULOS (Member), recalling the author's statement that where rubber tyred cranes were to be used the building would be simplified, asked to what extent, in a steelworks of the size of the Abbey Works or some others in Wales, (to what extent) it would be possible to eliminate piles.

Mr. LEWIS replied that in an ordinary building, with steel wheeled cranes, if the column head leaned over to the extent of one inch we knew we were beginning to get into trouble, and if it moved two inches we were *in* trouble. With rubber-tyred cranes we could have a column head moving over a little and rubber tyres would not notice it. The movement of the gantry track was no longer a determining factor in the decision whether or not to pile.

The proposed 120-ton hot slab yard Goliath crane with pneumatic tyred wheels, a model of which was exhibited at the meeting, was to run on a concrete strip foundation about 12 inches thick, on very weak ground. This strip was made up of short and long lengths hinged together alternately to break up the ' pot-hole ' effect, so that a wave form would not build up, as it would if the concrete strip were made up of uniform lengths. Even with the 120-ton crane the operating pressures underneath the track would be down to something under half a ton per sq. ft.

Mr. R. C. W. BEDFORD (Associate-Member), asked if any experience had been obtained of the effect of temperature on the rubber tyres. Gantry rails and gantry girders, he said, sometimes became quite uncomfortably hot, and he wondered if the rubber tyred wheels could be used in the same conditions.

Mr. LEWIS said that in the paper he had quoted temperature as the one limitation of the rubber tyres, and had mentioned 150° F. as the maximum working temperature. One could foresee the development of special tyres in the future for cranes alone ; when that

time came we might very well go to much higher temperatures.

The 150° F. represented a limiting factor and that was why the hot slab yard scheme he had mentioned did not go ahead. It had become necessary to change the scheme and to provide an overhead crane, and the use of rubber tyres had to be declined purely because of the temperature of the girders which was estimated at 250° F. Rubber tyres, however, can withstand short periods at very high temperatures. To illustrate this the figure shows a cupola drop car with pneumatic tyres. This carries away the red hot and part molten contents of an iron making cupola at the end of a day's work. The tyres had to run over liquid slag at every operating cycle and they caught fire. However, fortunately the next operation was to quench the whole car. Even in those conditions the tyres lasted about nine months.

Mr. P. M. WORTHINGTON asked for the author's opinion of the use of rubber wheels with steel flanges to guide them. Although this seemed a strange mixture, the author's formula (4) showed that the sideways force on the flange would only be a small fraction of the force with a traditional steel wheel. A flange need not be hard up against the side of the rubber wheel, but could be in the form of a disk placed at any convenient distance ; it could operate against the side of the top flange of the gantry girder.

Mr. LEWIS said there was a plan to do precisely that. There was a design at present where the intention was to run a rubber tyre with flanges on either side. The French Railways did that ten years ago.

One of the features of the rubber tyre was that it was almost completely silent ; the intention was to face the flange with tugnol or other material of that kind.

Mr. R. N. TAYLOR said it seemed to him that the rubber tyred type of crane was very useful, particularly for reinforced and prestressed concrete construction.

He asked about the lengths of crane girders and the range of overhead cranes.

Next he illustrated examples of the use of light alloy for cranes.

Finally he raised the question of moving the positions of the guide rollers, which might help in reducing the steering and other forces mentioned in the paper.

Mr. LEWIS in reply said that there was no reason at all why the guide rollers need be alongside the wheels. We could have the steering assembly running off an independent track.

He thought it a mistake to think of modifying existing cranes and putting on rubber tyres. We should regard the rubber tyred crane as something completely new, and he would prophesy that, if the rubber tyred crane development went well, in five years it would look very different from the cranes of to-day.

At the present stage we could say that rubber tyred cranes were worth while only where the cranes were an operating part of the plant. That was a fairly sweeping statement ; but in his view the sort of cycle for which a rubber tyred crane became worth while was at about 100 cycles per day. In steelworks particularly most of the cranes made between 125 and 150 moves per day.

Mr. WORTHINGTON said that the traditional crane with steel wheels generally has a long-travel shaft, and he thought that this was the main reason why the wheels had always been kept at about the same level as the crane bridge ; for otherwise four sets of bevel wheels would be required. With rubber wheels and independent drives, the long-travel shaft disappears and there is no longer any need to keep the wheels as high. This could be a great advantage in planning where the economic principle is often to keep everything as near ground level as possible. The height of the crab of the crane is determined just by the height the hook is required to go ; this in turn determines the height of the crane bridge ; but the height of the wheels, end carriages and gantry girders could be determined independently starting with the minimum head room required by plant or operations underneath the gantry girders. The difference between the heights of end carriages and crane bridge is then made up with structural legs to the crane.

Mr. B. L. CLARK (Member) said he really felt that Mr. Lewis had got something, though he did not quite know what. When one looked at vast earth moving machines dashing around on rubber tyres one felt some confidence in the tyres. When he had first heard Mr. Lewis talk about rubber tyres on cranes he was horrified, but he was not now ; the work seemed to be concerned principally with fairly light cranes. He asked if any good suspension element had been developed.

A number of the points raised in the paper did not necessarily apply to rubber tyre standards. To begin with why should we want to have faster cranes because they were on tyres ? He rather felt that the speed of the cranes was dictated by safety considerations. The men could not handle them too quickly or the process did not require the job to be done more quickly.

There had been one or two experiments on electric steering and they had been quite successful ; a crane could be steered electrically, whether it was on steel or rubber tyres.

The accelerometer interested him ; it was useful to be able to get all sorts of readings with it.

We could not dispel the forces of impact and vibration just because we used rubber tyres. It depended entirely on the natural frequencies of the two units.

Recalling some trouble which had occurred with some cranes about three years ago, Mr. Clark said that a few thousands of pounds had been spent in trying to put matters right. Then the sum of 30/- was spent on some molybdenum disulphide grease, and there had been no trouble since. He asked if Mr. Lewis had had anything to do with that type of material.

Mr. LEWIS commented that he agreed with almost everything Mr. Clark had said.

When he had referred to speed he had not meant necessarily that the crane was going to travel at 2,000 ft. per minute or something of that sort. But there were some very long buildings nowadays and the accelerating time with cranes was a large part of the operating time. The coefficient of friction of rubber on steel or concrete was up by a factor of three in the wet and about five in the dry, and in driving the same number of wheels we could increase the acceleration greatly. The limit was not the rubber tyre itself but the rather antiquated driving system. The operator had his eye on what was put on to the hook and he needed the same kind of three-dimensional control as did the pilot of an aeroplane. No doubt the B.I.S.R.A. could provide us with some information.

As to the speed at which we wanted a rubber tyred crane to travel, it rather depended on the influence the tyres would have. But he did not think very long buildings would be necessary when we had cranes that would go round corners and up hills. It might be that a very long building was very expensive ; for a given floor area the square building was very much cheaper than a very long one, and one could envisage buildings that were roughly square ; so that his feeling was that

a time of about one minute to get from one end of the building to another would be about right; by the time the load had been picked up and put down again that might be somewhere about the economic level.

Concerning the reference to molybdenum disulphide, he said when one started off with a system which came into use 150 years ago and developed it a little bit here and there, if finally the way to make it run steadily was to inject the best known lubricant at the nearest possible point to the place where the whole system relied on friction, then surely one was pushing the whole concept beyond its natural limit.

Mr. R. D. TEAGUE (Associate-Member), referred to a statement made by Mr. Lewis concerning a crane with horizontal rollers which had been erected in this country and on which during commissioning the rollers had fallen off. Mr. Teague stated that he assumed the crane referred to was one erected a few years ago by the Company that he is associated with. Actually seven cranes were installed and in no case did the horizontal rollers fall or become forced off during commissioning. Up to date these cranes are giving reasonable service.

Referring to another statement by Mr. Lewis that settlements and lateral deflexions of up to a foot could be allowed on crane gantries when horizontal rollers were used, Mr. Teague asked Mr. Lewis whether this dimension was rather excessive. Surely the allowable end clearances on the horizontal rollers should be the controlling factor in designing permissible settlements/deflexion.

Mr. Teague next asked whether impact allowances as specified in B.S. 449 could be reduced if rubber tyred runners, vertical and horizontal, were used.

Mr. LEWIS replied that if one were designing for the use of rubber tyred cranes a building subject to frequent foundation movements, as in a shipyard, where the gantry's tracks might move in phase with the tides, he thought the thing to do was to use electric steering, with a mechanised safety device. He felt it would be quite wrong to push the crane around with guide rollers to make it steer along a crooked track. One had noticed in the film that was shown that the guide rollers were not turning at all; and, in fact, in the buildings in which they were operating the cranes were true running. The rollers only touched when they were nudging a crane around a corner.

Concerning impact forces, Dr. Terrington and himself had collaborated in making some strain gauge measurements some years ago, and he wholly supported Dr. Terrington's figure of five per cent on the static load, with a good track.

As had been stated by Mr. Clark, the stress that occurred was a matter of natural frequencies of the units in the crane system; it was a matter of computation in each case. We could not say that because we were using rubber tyres we should be better off; we might be worse off. But Mr. Lewis made the point that whereas with a steel wheel the actual pressures at the points of contact were of the order of 40 tons per sq in., they dropped to the order of 100 lb. per sq. in., with pneumatic tyres and 200 with solid rubber tyres.

THE PRESIDENT, in thanking Mr. Lewis for the excellent way in which he had handled the discussion, said the response by way of discussion was a just reward to him for the work he had put into the paper, The meeting had been quite outstanding.

Institution Notices and Proceedings

PRESIDENTIAL ADDRESS

The Presidential Address for the Session 1961-62 will be given by Mr. F. R. Bullen, B.Sc. (Eng.), M.I.Struct.E., M.I.C.E., at 11, Upper Belgrave Street, London, S.W.1., on Thursday, 5th October, 1961, at 6 p.m.

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, 12th October, 1961.

Ordinary Meeting at 6 p.m., when a paper on "Professional Practice and Drawing Office Procedure" will be given by Mr. L. R. Creasy, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E.

Thursday, 26th October, 1961.

Ordinary General Meeting for the election of members at 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when a paper on "The Plastic Design of Lattice Portal Frames" will be given by Mr. B. H. Fisher, B.Sc., A.M.I.Struct.E., A.M.I.C.E., Dr. Jacques Heyman, M.A., A.M.I.C.E., and Dr. L. G. Jaeger, M.A., A.M.I.C.E.

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

EXAMINATIONS, JANUARY, 1962.

The Examinations of the Institution will next be held in the United Kingdom and overseas on the 9th and 10th January, 1962 (Graduateship) and the 11th and 12th January, 1962 (Associate-Membership).

ANNUAL DINNER, 1962

The Annual Dinner of the Institution will be held at the Dorchester Hotel, London, W.1., on Friday, 4th May, 1962.

PREPARATION FOR THE EXAMINATIONS OF THE INSTITUTION BY ATTENDANCE AT TECHNICAL COLLEGES

A candidate for Graduateship or Associate-Membership may be able to attend a technical college; these notes are intended to guide him in choosing the most suitable instruction.

PREPARATION FOR THE GRADUATESHIP EXAMINATION

Technical Colleges offer :—

(a) Full-time or sandwich courses for degrees or for Higher National Diplomas or College Diplomas in Building or Engineering.

(b) Part-time day or evening courses for Higher National Certificates in Building or Engineering

If he obtains a Higher National Certificate or Diploma complying with the exempting qualifications of the Regulations Governing Admission to Membership, the candidate will be exempted from the Graduateship Examination.

Alternatively, he may study subjects selected from the available courses and sit the Graduateship Examination. At technical colleges, courses are usually available in Mathematics, Building Science or Engineering Science, Strength of Materials, Theory of Structures, Surveying and Engineering Geology, but students are not normally allowed to select subjects from National Diploma or Certificate courses unless they can show evidence of sound training in more elementary studies. The advice of the college authorities should be followed.

PREPARATION FOR THE ASSOCIATE-MEMBERSHIP EXAMINATION

At some technical colleges there are part-time courses in Structural Engineering which cover the syllabus of the Associate-Membership Examination. At other colleges the candidate must rely on Higher National Certificate courses or on advanced courses in Building or Civil Engineering covering only part of the requirements for the Associate-Membership Examination.

Colleges in List 'A' provide at least two years of instruction in Theory of Structures and in Structural Engineering Design and Practice up to Associate-Membership standard.

LIST 'A'

Bath Technical College.
Belfast College of Technology.
Birmingham College of Advanced Technology.
Bolton Technical College.
Bradford Institute of Technology.
Bridgend Technical College.
Bristol College of Technology.
Chesterfield College of Technology.
Coatbridge Technical College, Lanarkshire.
Derby and District College of Technology.
Dudley and Staffordshire Technical College.
East Ham Technical College, E.6.
Enfield Technical College.
Ewell County Technical College.
Glasgow Royal College of Science and Technology.
City of Liverpool College of Building.
L.C.C. Brixton School of Building, S.W.4.
L.C.C. Hammersmith College of Art and Building.
Manchester College of Science and Technology.
Middlesbrough, Constantine Technical College.
Nottingham and District Technical College.
Portsmouth College of Technology.
Salford, Royal Technical College.
S.E. London Technical College, Worseley Bridge Road, S.E.26.
S.W. Essex Technical College, Walthamstow, E.17.
Southampton Technical College.
Stafford County Technical College.
Stockport College of Further Education.
Tottenham Technical College.
Twickenham Technical College.
Willesden Technical College, N.W.10.

Colleges in List 'B' provide instruction in Theory of Structures to Associate-Membership standard, but instruction in Structural Engineering Design and Practice is not usually so complete.

LIST 'B'

Brighton Technical College.
Cardiff Technical College.
Edinburgh, Heriot-Watt College.
Huddersfield Technical College.
Leeds College of Technology.
London, Battersea College of Technology, S.W.11.
London, Northampton College of Advanced Technology, E.C.1.
L.C.C. Westminster Technical College, S.W.1.
Newcastle upon Tyne, Rutherford College of Technology.
Plymouth and Devonport Technical College.
Preston Harris Institute.
Rotherham College of Technology.
S.E. Essex Technical College, Dagenham.
Wigan Mining and Technical College.
Woolwich Polytechnic, S.E.18.
West Ham College of Technology.

Students are advised to take the organised courses in Structural Engineering where these are available.

FAIR COPYING DECLARATION

The Institution of Structural Engineers, in common with other scientific and learned societies, subscribes to the Royal Society's Fair Copying Declaration, which reads as follows :

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SOME RECENT PUBLICATIONS OF INTEREST TO STRUCTURAL ENGINEERS

BRITISH STANDARDS INSTITUTION

B.S. 466 : 1960. Specification for Electric Overhead Travelling Cranes for General Use in Factories, Workshops and Warehouses. 10s.

B.S. 2573 *and B.S.* 466 : 1960. Two new revised Standards for Cranes. 12s. 6d. and 10s. respectively.

B.S. Code of Practice. CP.3, Chapter 3 (1960). Code of Basic Data for the Design of Buildings. (Formerly Code of Functional Requirements of Buildings). Sound Insulation and Noise Reduction. The Council for Codes of Practice, British Standards Institution. 20s.

CEMENT AND CONCRETE ASSOCIATION

C.&C.A. Technical Report TRA/343, November, 1960. "A Preliminary Examination of the Vebe Consistometer" by P. W. Keene, B.Sc.(Eng.), A.M.I.C.E.

DEPARTMENT OF SCIENTIFIC AND INDUSTRIAL RESEARCH

Road Research. Road Note No. 29. A Guide to the Structural Design of Flexible and Rigid Pavements for New Roads. D.S.I.R. Road Research Laboratory, H.M.S.O. 1960. 2s.

Research and Development Requirements of the Shipbuilding and Marine Engineering Industries. December, 8th, 1960. (47-227). 1s. 8d.

Greenford Roundabout Experiment, by F. V. Webster, B.Sc., Ph.D., Road Research Laboratory, D.R.I.R. Reprint from Traffic Engineering and Control, 1960, 2 (5) (266-271).

Changes in Accident Frequency after Changes in Speed Limits in the United Kingdom, by F. Garwood, Road Research Laboratory, D.S.I.R., and J. T. Duff, Ministry of Transport, Great Britain. Theme VI. The Influence of Speed and Speed Regulations on Traffic Flow and Accidents.

Road Abstracts. Vol. XXV 1958, Index (72-25-58-13), 2s. 10d. Vol. XXVII 5th May, 1960, Abstracts Nos. 398 to 503 (72-25-60-5), 2s. 8d. 6th June, 1960, Abstracts Nos. 504 to 615 (72-25-60-6), 2s. 8d.

Road Note No. 26. Guide to Engineers on the Making Up of Private Streets (47-141-26). 1s. 2d.

Technical Paper No. 48. Assessment of priority for Road improvements, by D. J. Reynolds, B.Sc.(Econ.), June, 1960 (47-110-48). 2s. 10d.

Building Research Station Digest No. 136. July, 1960. Index to Digests 1-135 (72-22-1-36). 6d. Quantity rates on application.

Digest No. 5. (Second Series). Materials for Concrete (72-36-0-5). 6d.

Building Science Abstracts. Vol. XXXIII, 2nd February, 1960, Abstracts Nos. 167-350 (72-4-60-2), 3s. 4d. 3rd March, 1960, Abstracts Nos. 351-527 (72-4-60-3), 3s. 4d. 4th April, 1960, Abstracts Nos. 528-702 (72-4-60-4), 3s. 4d. May, 1960, Abstracts Nos. 703-882 (72-4-60-5), 3s. 4d. June, 1960, Abstracts Nos. 884-1060 (72-4-60-6), 3s. 2d. July, 1960, Abstracts Nos. 1061-1211 (72-4-60-7), 3s. 4d. August, 1960, Abstracts Nos. 1212-1373 (72-4-60-8), 3s. 4d. September, 1960, Abstracts Nos. 1374-1556 (72-4-60-9), 3s. 4d.

Building Research 1959. Report of the Building Research Board with the report of the Director of Building Research (47-61-0-59). 7s. 6d.

DRURY MEDAL AWARD—1961

The alternative subjects for this the eighth Competition, are a pipe-bridge and a petrol service station.

Graduates and Students of the Institution not over 27 years of age are invited to apply for full details to the Secretary, envelope to be marked in the top left-hand corner 'Drury Medal Award.'

The closing date for the competition is October 1st, 1961.

The general conditions of the competition are as follows :

(a) The competition shall be for Graduates and Students of not more than 27 years of age.

(b) The subjects of the competition will be designs of a structural character, that is to say, involving structural design rather than planning.

(c) The subjects of design and the conditions shall be prepared and issued biennially.

(d) A Jury shall be appointed to examine the work submitted and to interview candidates if found necessary.

(e) In order to ensure that the design submitted is the unaided work of the competitor, the drawings, calculations, etc., submitted shall be endorsed by the candidate : "I declare that the work I hereby submit is my own unaided work." The declaration shall be signed by the competitor and be either countersigned by a corporate member, or be certified as made before a Justice of the Peace, or a Commissioner for Oaths.

Branch Notices

LANCASHIRE AND CHESHIRE BRANCH

The opening meeting of the Session will be held on Tuesday, 17th October, 1961, at the College of Science and Technology, Manchester, when Mr. J. B. Story, M.I.Struct.E., will give the Chairman's Address. The meeting will commence at 7 p.m., and will be preceded by a Hot Pot Supper.

Joint Hon. Secretaries : W. S. Watts, M.I.Struct.E., A.M.I.C.E., 11, Newchurch Lane, Culcheth, Nr. Warrington, Lancs. M. D. Woods, A.M.I.Struct.E., 8, Dennison Road, Cheadle Hulme, Cheshire.

MIDLAND COUNTIES BRANCH

The following Honorary Officers and members of the Committee have been elected for the Session 1961-62 :—

Chairman : J. E. C. Farebrother ; *Vice-Chairmen* : L. A. Firminger and S. M. Cooper ; *Hon. Treasurer* : W. Phillips ; *Hon. Auditors* : M. D. Picknell, and A. R. Moseley ; *Hon. Secretary* : A. N. Jones ; *Hon. Assistant Secretary* : S. J. Condliffe ; *Hon. Assistant Secretary* : R. H. Harrison ; (Derby Area). *Hon. Assistant Secretary* : C. B. Brewington ; (Nottingham Area). *Committee Members* : R. D. Anchor, J. R. Chaffer, E. R. Deeley, R. H. Howl, H. E. Thrower, E. E. Booker, K. L. Bourne, D. T. Bullock, S. G. Moore, F. W. Hurle, W. J. N. Mayo, R. D. Teague. *Hon. Secretary* : S. M. Cooper, M.I.Struct.E., "Applegarth," Hyperion Road, Stourton, Stourbridge, Worcs.

GRADUATES' AND STUDENTS' SECTION

Hon. Secretary : H. T. Dodd, A.M.I.Struct.E., Shepherd's Cottage, Grove Lane, Wishaw, Sutton Coldfield, Warwicks.

NORTHERN COUNTIES BRANCH

A meeting will be held at the Cleveland Scientific and Technical Institution, Middlesbrough, on Thursday, 12th October, 1961, when an Address will be given by the Chairman-Elect, Mr. L. Dobson, M.I.Struct.E.

A meeting will be held at the Neville Hall, Newcastle, on Monday, 16th October, 1961, when the President of the Institution, Mr. F. R. Bullen, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E., will install the Branch Chairman, Mr. L. Dobson, M.I.Struct.E., who will then give the Chairman's Address for the Session 1961-62.

Both meetings will commence at 6.30 p.m., preceded by buffet tea at 6 p.m.

Hon. Secretary: P. D. Newton, B.Sc. (Eng.), M.I.Struct.E., A.M.I.C.E., 6, Cornfield Road, Linthorpe, Middlesbrough, Yorks.

NORTHERN IRELAND BRANCH

The opening meeting of the Session will be held on Tuesday, 31st October, 1961, in the David Keir Building, Civil Engineering Department, Queen's University, Belfast, when a paper on "The Structural Design of Merchant Ships" will be read by Mr. J. Williamson, B.Sc., A.M.R.I.N.A. The meeting will commence at 6.30 p.m., and tea will be served from 5.45 p.m.

Hon. Secretary: L. Clements, A.M.I.Struct.E., A.M.I.C.E., A.M.I.Mun.E., 3, Kingswood Park, Cherryvalley, Belfast.

SCOTTISH BRANCH

Hon. Secretary: W. Shearer Smith, M.I.Struct.E., A.M.I.C.E., c/o The Royal College of Science and Technology, George Street, Glasgow, C.1.

SOUTHERN BRANCH

The opening meeting of the Session will be held on Friday, 10th November, 1961, in the Main Lecture Theatre, Lanchester Building, University of Southampton, when Mr. J. R. Lowe, B.Sc., M.I.Struct.E., A.M.I.C.E., will give the Chairman's Address. The meeting will commence at 6.30 p.m.

Hon. Secretary: A. P. K. Tate, B.Sc.(Eng.), Ph.D., A.M.I.Struct.E., Department of Civil Engineering, The University, Southampton.

WALES AND MONMOUTHSHIRE BRANCH

The following Honorary Officers and members of the Committee have been elected for the Session 1961-62 :—

Chairman: E. Hughes; *Vice-Chairman*: K. J. Stewart; *Hon. Secretary*: W. D. Hollyman; *Hon. Assistant Secretary*: E. R. Stewart; *Hon. Auditors*: J. M. Burke and D. A. Lewis.

Ordinary Members of Committee: D. Manolopoulos; W. A. Evans; G. R. Brueton; A. Bone; J. M. Burke; G. E. Cooper; D. H. Corlett; J. D. Davies; M. J. M. Hares; A. V. Hooker; G. Hughes; M. J. Harvey; D. A. Lewis; G. W. Spooner; E. R. Steward; A. G. Thompson.

Ex officio Members: E. Hughes, W. D. Hollyman, K. J. Stewart.

The opening meeting of the Session will be held on Tuesday, 24th October, 1961, at the South Wales Institute of Engineers, Park Place, Cardiff, when the Chairman's Address will be given by Mr. E. Hughes, M.I.Struct.E. The meeting will commence at 6.30 p.m.

Hon. Secretary: W. D. Hollyman, A.M.I.Struct.E., 41, Greenfield Avenue, Dinas Powis, Glamorgan.

WESTERN COUNTIES BRANCH

The opening meeting of the Session will be held on Friday, 13th October, 1961, in the Small Lecture Theatre, Queen's Building, University Walk, Bristol. Mr. Clifford E. Saunders, M.I.Struct.E., will give the Chairman's Address and the presentation of the Branch Prize and the Francis Memorial Prizes will be made. The meeting will commence at 6.30 p.m., and will be preceded by a light tea at 5.30 p.m.

Hon. Secretary: M. S. G. Cullimore, B.Sc., Ph.D., A.M.I.Struct.E., Queen's Building, University Walk, Bristol, 8.

YORKSHIRE BRANCH

The opening meeting of the Session will be held at the Metropole Hotel, Leeds, on Wednesday, 18th October, 1961, when Mr. T. E. S. White, B.Sc., M.I.Struct.E., M.I.C.E., will give the Chairman's Address. The Address will be repeated at the Royal Victoria Hotel, Sheffield, on Thursday, 19th October. Both meetings will commence at 6.30 p.m., preceded by buffet tea at 6.15 p.m.

Hon. Secretary: W. B. Stock, A.M.I.Struct.E., 34, Hobart Road, Dewsbury, Yorks.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary: E. B. Kretzschmar, A.M.I.Struct.E., P.O. Box 3306, Johannesburg, South Africa.

Natal Section Hon. Secretary: J. C. Panton, A.M.I.Struct.E., A.M.I.C.E., c/o Dorman Long (Africa) Ltd., P.O. Box 932, Durban.

Cape Section Hon. Secretary: R. F. Norris, A.M.I.Struct.E., African Guarantee Building, 8, St. George's Street, Cape Town.

SECTION NOTICES

AUCKLAND (NEW ZEALAND) SECTION

Hon. Secretary: A. Donald, B.Sc.(Hons.), A.M.I.Struct.E., 122, Matipo Road, Te Atatu, Auckland, New Zealand.

EAST AFRICAN SECTION

The following Honorary Officers and members of the Committee have been elected for the Session 1961-62 :—

Chairman: J. A. K. Schou; *Vice-Chairman*: S. Geleff; *Hon. Secretary*: K. C. Davey, A.M.I.Struct.E., P.O. Box 30079, Nairobi, Kenya.

Committee: R. A. Sutcliffe, S. L. Sunnak, S. C. Aggarwal, J. Erby, H. Cuerden.

NIGERIAN SECTION

Hon. Secretary: A. Brimer, M.I.Struct.E., Brimer, Andrews & Nachshen, Private Mail Bag, 2295, Lagos, Nigeria.

REPUBLIC OF IRELAND SECTION

Hon. Secretary: P. J. Carroll, M.E., A.M.I.Struct.E., A.M.I.C.E., A.M.I.C.E.I., 9, Laburnum Road, Clonskea, Dublin, Ireland.

SINGAPORE AND FEDERATION OF MALAYA SECTION

Hon. Secretary: J. R. M. MacIntyre, A.M.I.Struct.E., c/o Redpath, Brown & Co. Ltd., P.O. Box 648, Singapore.

SOUTH WESTERN COUNTIES SECTION

The opening meeting of the Section will be held at the Grand Hotel, The Hoe, Plymouth, on Friday, 3rd November, 1961, at 6.30 p.m., when Mr. E. G. Cove, B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., will give the Chairman's Address.

Hon. Secretary: C. J. Woodrow, J.P., M.I.Struct.E., "Elstow," Hartley Park Villas, Tavistock Road, Plymouth, Devon.